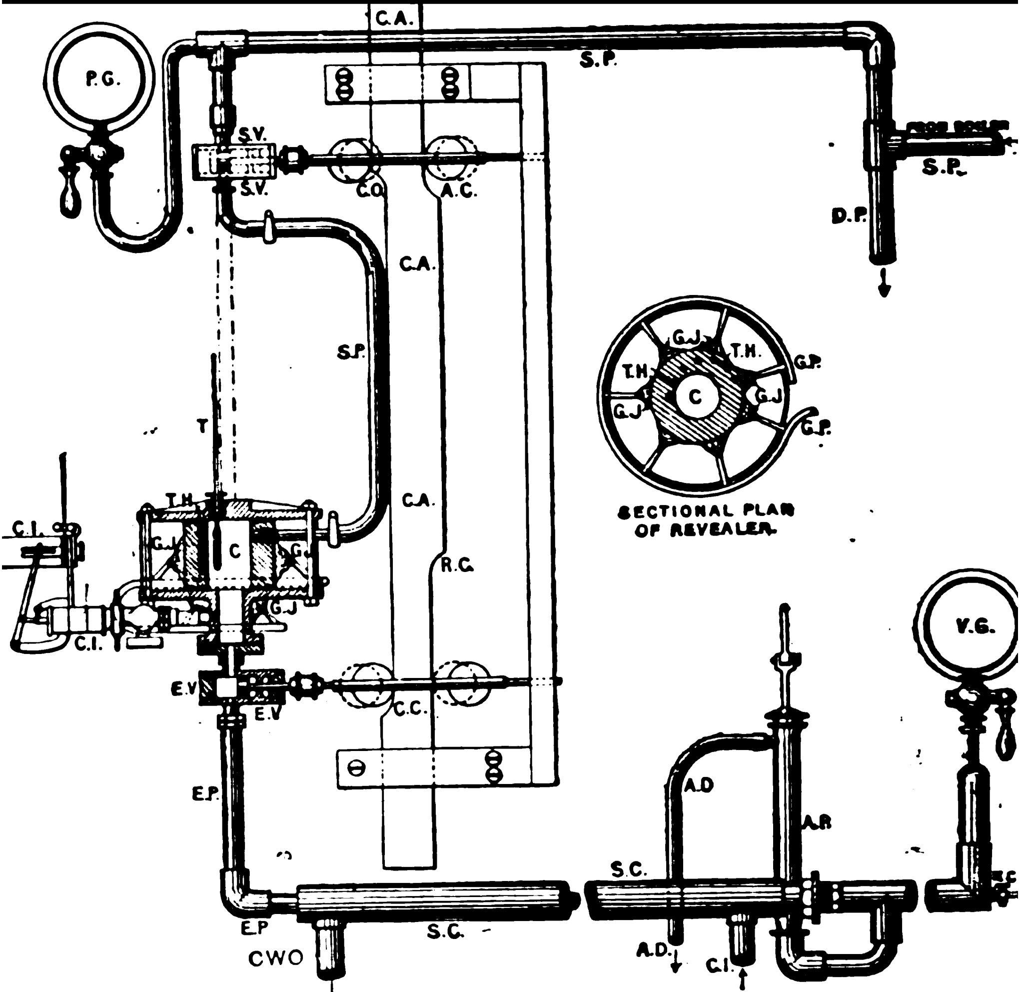

Minutes of proceedings of the Institution of Civil Engineers

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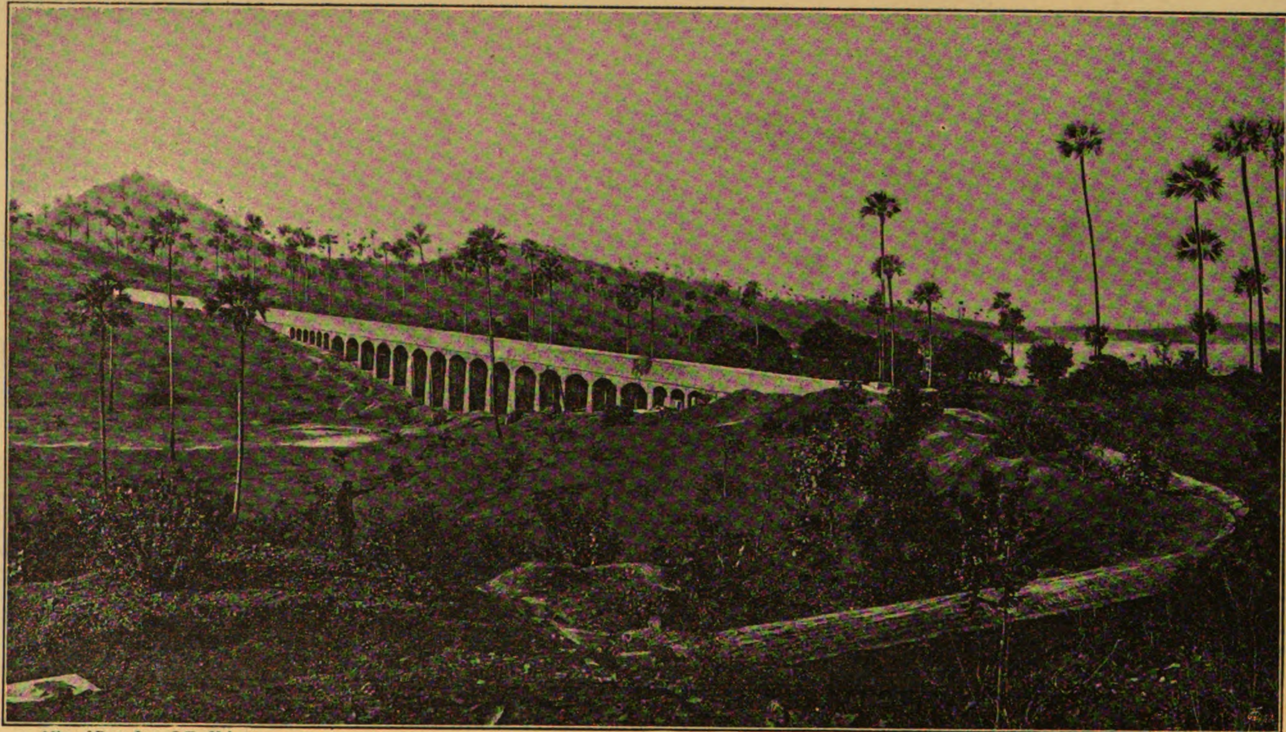

Min. of Proc. Inst. C.E., Vol. cxv. T. Kell & Son.

VIEW OF THE BHANDOOP AQUEDUCT, TANSA (BOMBAY) WATERWORKS

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MINUTES OF PROCEEDINGS

London

OF

THE INSTITUTION

OF

CIVIL ENGINEERS;

WITH OTHER

SELECTED AND ABSTRACTED PAPERS.

VOL. CXV.

EDITED BY

JAMES FORREST, Assoc. Inst. C.E., Secretary

LONDON:
Published by the Institution,
25, GREAT GEORGE STREET, WESTMINSTER, S.W.
[Telegrams, "Institution, London." Telephone, "3051."]
1894.

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LONDON: PRINTED BY WM. CLOWES AND SONS, LIMITED, STAMFORD STREET AND CHARING CROSS.

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CORRIGENDA.

Vol. lvii. p. 9, footnote, *for* "171," *read* "121."
Vol. cxiv. p. 408, line 20, *for* "laying," *read* "lying."
" 410, line 26, *for* "is," *read* "are."
" 450, line 18 from bottom, *for* "sandstone," *read* "fireclay."
" 477, line 8, *for* "are," *read* "is."

THE INSTITUTION OF CIVIL ENGINEERS.

SESSION 1893-94.—PART I.

SECT. I.—MINUTES OF PROCEEDINGS.

14 November, 1893.

Sir ROBERT RAWLINSON, K.C.B., Vice-President, in the Chair.

Sir ROBERT RAWLINSON, the senior Vice-President, who occupied the chair in the absence of Mr. Giles, the President, through indisposition, rose—an act which was followed by all present—and, having alluded to the great loss the Institution had sustained by the death, during the recess, of its old and highly-esteemed Past-President, Mr. Thomas Hawksley, stated that the Council had conveyed to the family on their behalf, and that of the members generally, the assurance of their sincere condolence.

The Presidential Address of Mr. Giles was then read, as under:—

GENTLEMEN,—

When, at the close of the last session, you did me the honour of electing me to occupy the Presidential chair for the ensuing year, I found that, by some old-fashioned practice, it was not usual to return thanks at the time of election, although the custom is contrary to all precedent in the case of ordinary elections. I therefore take advantage of this occasion to return you my most cordial thanks for the honour you have conferred on me; and I promise to do my best to follow in the steps of my worthy predecessors, and to the utmost of my power to maintain the prestige and promote the interests of the Institution. I must, however, remind you that I am the senior of all previous Presidents at the time of their election, and on this ground alone I may claim indulgence if at any time I fail in my responsible duties for lack of energy.

After reviewing the Presidential addresses for several years past, I have come to the conclusion that the problem of broaching a new subject to the engineering world has become each year

more and more difficult. You have already had the history of nearly all the great works that have been designed and executed up to the present time, not only in this country, but in all parts of the world, comprising railways, docks, harbours, canals, bridges, waterworks, drainage, reclamations, breakwaters, tunnels, electric cables, locomotives, steam-ships, engines of all descriptions, and lastly, the manufacture of guns for warfare and national defence. I do not propose, therefore, to recount the various triumphs of engineering skill that are constantly swelling the list, but to give a short account of my personal experience during the past sixty years, and to mark the progress made in the rapidity and comfort of travelling at the present time compared with what it was in my early days.

In speaking of those days, I may refer to my first long journey in 1831 from London to Newcastle—280 miles. This distance was accomplished by mail-coach in thirty hours, including stoppages at Grantham and York for breakfast and dinner; but the discomfort of four persons sitting inside a small coach for thirty hours, cramped and half choked by foul air, may be better understood than described. I might also speak of another journey from Carlisle to London—300 miles—in 1832, on the outside of the coach, when the passengers had to endure the misery of getting half wet through in a snow-storm on Shap Fell; and it was not till the year 1836, when coaches had attained their highest speed of 10 to 11 miles an hour, including stoppages, that they began to be seriously interfered with by the progress of railways.

In the year 1825, the first railway line (Stockton and Darlington) was opened for passengers, and in the year 1838 the London and Birmingham Railway was opened as far as Denbigh Hall, some 40 miles from London. From that time coaches were gradually driven off the road by the greater economy, superior comfort, and higher speed of journeys performed by rail; and it is difficult to realize now the great difference between the safety and luxury of travelling at the present time compared with that of sixty years ago.

The safety of railway travelling compared with that of coaching appeals to me, for I was once on the box of a mail-coach when the driver, having lost all control of his horses, asked me to take the reins of the leaders and pull at them. I might as well have pulled at a house, and I had the pleasure of being run away with for 4 miles on a dark night, until the horses stopped from sheer exhaustion. On this occasion there was fortunately no accident, but from my recollection of the frequency of coach accidents in those days, I believe that if statistics of these could be obtained

they would prove the safety of railway-trains to be infinitely greater than that of the old coaches—the railway accidents of last year showing that only one out of about every seven millions of railway passengers lost his life from all causes. The luxury of travelling at the present time in the best trains on the best lines need scarcely be mentioned; particularly in the trains to the North and to Scotland, the accommodation provided is as good as that in an hotel, and at a cost which is within the reach of all classes. A comparison of the third-class carriages of the present day with those of the earliest types, will show that the latter were no better than pig-trucks, without shelter from the weather, whilst the former are better than the early first-class carriages.

In the early days, before the penny post was established in 1840, the postage for a single letter to Newcastle was 1*s.*; to Mortlake it was 4*d.*, and the lowest charge for any delivery in London was 2*d.* Envelopes had not then been invented, every letter being written on a sheet of paper, and it mattered not whether the sheet was of note size or of foolscap; but if the smallest cutting from a newspaper, or any additional slip was inserted in a letter, and it was discovered by the Post Office, that letter was charged double postage. This was the custom in what are commonly called the "good old times," and under such regulations it is not difficult to understand that in 1831 two mail-coaches daily were sufficient to carry the whole of the mails from London to Scotland—one going through Newcastle to Edinburgh, the other through Carlisle to Glasgow.

Compared with this, it is scarcely possible to believe that the enormous development of trade in all parts of Great Britain and Ireland has resulted in the construction of over 20,300 miles of railway, and of a train-service of 328,000,000 miles a year. The cost of these 20,300 miles of railway largely exceeds the amount of our National Debt; and the gross returns of the traffic, involving the carriage of 900,000,000 passengers, of 310,000,000 tons of goods, minerals, &c., and of about two billions and a half (2,500,000,000) letters and parcels, amount to a sum no less than £82,000,000 a year.

These changes were not made rapidly, nor without taxing to the utmost the skill and energies of the greatest engineers; and I make bold to say that the world at large is more indebted to the labours of engineers for the progress of civilization than to those of any other profession.

There is perhaps not much room in this country for any great extension of the railway system, but having regard to the enor-

mous cost at which many lines have been carried through house property in our great cities, it is quite possible that the construction of underground railways may in the future occupy more attention than it has up to the present time received. There is, however, ample scope for the energies of engineers abroad, particularly in India and in the colonies. Notwithstanding the vast area of India, which is twenty times larger than this country, there are in that country only 20,000 miles of railway open, representing a capital of about £200,000,000, upon which expenditure a good return is made; while in the Cape Colony, with an area two and a half times larger than that of Great Britain, there are only about 2,200 miles of railway completed. As an encouraging feature towards future extension of these lines, I may, however, mention that the 2,200 miles of State railways executed in the Cape Colony alone yield a net return of about $4\frac{3}{4}$ per cent. on their cost. We must not, however, shut our eyes to this fact, that, having taught the world the art of railway construction, we will in future have to submit to competition with foreign engineers in all countries; and it is well that young engineers in this country have now facilities for study and improvement which were unknown to the older members of the profession.

It is not very easy to understand why English engineers, with all their dearly-bought experience, should have been parties to the design and construction of railways in India with no less than five different gauges. One would have thought the hard-fought battle of the gauges in 1846 would have convinced them of the folly of reverting to what was then so universally condemned. It is easy to understand the necessity of having one gauge for main and trunk lines, with a more economical system of branch lines for light traffic; but it would have been well if the Indian Government had interfered to stop any further deviation from the break of gauge, and had encouraged as far as possible the system of uniformity of gauge, which is so necessary for the development of railway traffic.

To sum up the progress of travelling by land during the last sixty years, I need only add that, at present, our fastest trains cover fully 46 miles an hour on the journey, including stoppages, as against 10 to 11 miles per hour by coach; and, as a locomotive has been proved to be able to run at the rate of 102 miles an hour, it would be unsafe to hazard an opinion that the greatest speed of railway travelling compatible with safety has yet been attained. I have the highest authority for stating that, provided the rolling-

stock and road are properly maintained and kept in good order, there is no undue risk in running at 84 miles an hour. Indeed I have travelled at the rate of 72 miles an hour, but must confess I did not like it. Nor is it safe to hazard a prophecy that the railway companies will be always content to be dependent solely on coal for power, as the adaptation of electricity and the increasing use of liquid fuel forbid any such supposition; any system would be welcomed which would have the effect of checking the tyranny of trades unions, as exemplified in the recent coal strikes, by which so many trades have been paralysed and driven out of the country.

Before concluding these remarks on the past history of railways, I would like to call attention to the change that has taken place with regard to the laying out of railway lines. During the five years between 1833 and 1838, I was almost exclusively engaged in levelling over the greater part of England for the purpose of finding the most suitable lines for the railways that were then being projected; and, instead of being welcomed by the landowners and farmers as their best friend, in being instrumental in increasing the value of the land through which the railways would pass, I was frequently driven off the ground *vi et armis*. On one occasion (which I well remember), I was threatened with the horsewhip by a burly farmer whose lands I had just crossed at four in the morning. I was only a stripling at the time, but having four chain-men and staff-holders with me, I was bold enough to tell the good man that if he laid his hands on me I would order my men to duck him in the nearest horsepond. The farmer thought better of his threat, and I merely relate this little anecdote to show how different the treatment was then from what would be now accorded to any engineer laying out a new railway-line through a new country.

I do not think the history of improved locomotion by land should be dismissed without an allusion to the wonderful progress made in the use and manufacture of cycles. In fact, this style of locomotion has become not only fashionable but eminently useful, and the keen competition between accomplished cyclists is such that a record has been made of a run of 433 miles in twenty-four hours—a feat which speaks volumes for the perfection of the cycle and for the endurance of the cyclist.

Now, if we have to congratulate ourselves on the vast improvement effected in travel by land during the past sixty years, we may also look back with pride upon what has been accomplished on the water. In 1819 the first steamer crossed from Holyhead to Dublin, a distance of 63 miles in about seven-and-a-half hours.

Previous to this the voyage was made by sailing-boats, and in bad weather and during calms or adverse winds, it is remembered that there has been no communication with Ireland for a fortnight, It is within my recollection that the trips across St. George's Channel were sometimes made with only one passenger.

What have we now? A mail service twice a day between Holyhead and Dublin, with others between Liverpool, Glasgow, Fleetwood, Stranraer, Bristol, Milford and various parts of Ireland. The Holyhead boats are perhaps as near perfection as may be, frequently accomplishing the passage of 63 miles under three-and-a-half hours; so that whatever reason there might have been in the early part of this century for not considering Ireland as part of the United Kingdom, that reason will not hold good now.

From this vast improvement in the communication between this country and Ireland, we may well turn to our steam-services with America, the Cape, India and Australia. It is just fifty-five years since the first steamboat started from Bristol to New York, and, notwithstanding the most dismal prophecies of failure, she made the voyage in 14½ days. Of course this success led to a rapid increase of vessels built for the American service; and in 1860, the Great Eastern, the magnificent creation of Brunel, 680 feet long, 83 feet beam, and of about 19,000 tons burthen, made the run from Halifax to Milford in a little over seven days. This vessel, in which I was a passenger, was so light from want of cargo, that she only drew 19 feet, and she was therefore detained at Halifax for the purpose of letting out the paddle-floats so as to get a better grip of the water, otherwise the passage home would have been given from New York. It was, however, found that she was too large for commercial purposes, and after having cost nearly a million sterling, she was laid aside, until bought by a company for the laying of submarine cables; and after having done some few years of profitable work in this somewhat limited service, she was sold in 1888 for £16,000 to be broken up.

What are we doing now, after a period of only thirty-three years? We are again approaching the dimensions of the Great Eastern, by the construction of the vessels Campania and Lucania, 620 feet in length, 65 feet in breadth, and of nearly 13,000 tons burthen, which have lately made the passage both to and from New York, in little more than five-and-a-half days.

Touching the economy of working the vessels of our mail services and mercantile marine, there is as yet no sign of finality in this direction having been approached. When we were content with a boiler pressure of 20 to 30 lbs., and a coal-con-

sumption of about 5 lbs. to 10 lbs. per I.HP., freights were sufficiently high to make steamships pay; but now that competition has reduced freights to starvation prices, the necessity for the most stringent economy has had the effect of encouraging improvements in all directions; and it is reasonable to expect that some other power than steam may be utilized, and some other fuel besides coal, by which further economies may yet be obtained. We now work at 180 and 200 lbs. pressure, with a coal-consumption reduced to about 1½ lb. per I.HP. When the idea of working at such high pressures was first entertained, it was thought that it would be very unsafe to subject a vessel to the risk contingent thereon; but as all boilers should be, and are generally, tested to and proved capable of sustaining double their working-pressure, it follows that a boiler of 200 lbs. working-pressure has a reserve of 200 lbs. of extra strength as against the smaller reserve of a boiler working at lower pressure.

Another element of safety in steamers having to make long ocean voyages is the adoption of twin screws, whereby if an accident occur by which one engine is disabled, the other may be trusted to bring the vessel safe into port. This has already happened in the Cape mail-service, and when voyages are made which cover a run of some 5,000 miles of open sea from port to port, this additional safeguard against accident at sea is not to be lightly disregarded. The accident to the thrust-block of the Umbria last year is a good case in point. Another reason for the adoption of twin screws is that vessels of large size and of great speed require very large screws and screw-shafts, the forging of which are much more difficult to obtain than those of smaller dimensions; the twin screws being also smaller than the single screw, do not work so near the surface of the water. Moreover, a vessel 400 feet long can be turned round in her own length by the twin screws being revolved in opposite directions. Against these advantages possessed by twin screws, it must not be forgotten that they require more attention, and, consequently, a rather larger staff in the engine-room.

The floating palaces of the present day have been created by the increasing demands of commercial enterprise, necessitating also an increasing number of travellers to and from all parts of the world; and the largest steamship companies have found it to their advantage to follow in the steps of the railway companies, and to provide accommodation for third-class passengers which is little inferior to that provided for first- and second-class. The most modern steamships are now provided with electric light, and

with refrigerating chambers from which the daily supply of fresh meat, fish, vegetables and fruit is furnished to their passengers; and by means of these chambers the importation of fresh fruit as well as of fresh meat from our colonies and from abroad, has developed into a very large trade; indeed, it would be difficult to feed our increasing population were it not that nearly the half of the steam tonnage of the world is in the hands of British owners.

Owing to the increased size of ocean steamships, the old-established docks are quite incapable of accommodating them, and there are therefore sundry new docks built and in course of construction to meet this want, notably at London, Tilbury, Liverpool, Glasgow, Hull, Cardiff, Barry and Southampton. At the latter port, when the first dock was designed, nearly sixty years ago, the lock-entrance was built only 46 feet wide. It was, however, widened to 60 feet more than forty-three years ago, and two or three years later a graving-dock with 80-foot gates was constructed for the Dock Company while I was their engineer; but since that Company was bought up, last year, by the London and South-Western Railway Company, there have been commenced an extension of deep-water quays, the largest dry dock in the world, and a channel 30 feet in depth at low-water of spring-tides right up from the sea to the new works. The outcome of these improvements has been to encourage and develop a large and increasing trade between America and Southampton; and the ample resources of the London and South-Western Railway Company have enabled them to provide all the most modern appliances to facilitate the rapid loading and discharging of vessels.

The heavy capital cost of large steamships renders it absolutely necessary that, if they are to be made to pay, they should be kept as short a time as possible in dock; and for this purpose every facility should be given for getting into dock (independently of tides), and for berthing, loading, and discharging. In the matter of berthing, I am of opinion that docks should be planned so as to avoid the necessity of having to swing ships round at right-angles, and for this purpose the Empress Dock at Southampton was designed in the shape of a diamond, so that vessels could berth themselves without having to turn, and to enable railway lines to be laid round the obtuse angles of the dock without the introduction of turntables, leaving the acute angle at the end of the diamond for the connection with outside railways.

When long voyages across the Atlantic, to the Cape, to India, and to Australia are reckoned by minutes, and are performed by

vessels costing hundreds of thousands of pounds, shipowners will not tolerate the waste of time caused by insufficient depth of water, or by want of the best and most complete machinery for expediting the movements of their ships.

A vessel has just been launched from Messrs. Harland and Wolff's ship-yard, at Belfast, which is to carry 14,100 tons outward, and to bring home 9,400 tons of cargo and coal, as well as 1,200 bullocks. She is to leave Liverpool on a Friday, arrive in New York in ten days, discharge her cargo, load again, and leave New York in seven days more, returning to Liverpool so as to be ready to start again to New York with a fresh cargo on the thirty-fifth day from her first departure from Liverpool. If this can be accomplished, it shows how necessary it must be for docks to be always easy of access, and to be furnished with the very best appliances for saving time in loading and discharging vessels. The Dock Trustees of Liverpool, having succeeded in considerably deepening the water over the bar of the Mersey, are prudently continuing their efforts in that direction, so as to make the channel to the docks available for ordinary vessels at all times of the tide.

In this country, although the construction of docks and railways must be somewhat limited in future, there is ample scope for the practice of engineers in sanitary works, water-works and the development of the applications of electricity. In fact, so great has been the demand for electric lighting, &c., to which is due the numerous applications everyone receives from self-styled electrical experts to enlighten their customers at the smallest possible cost, it may well be imagined that as almost every other man is an electrical engineer, the crop has sprung up rather too suddenly to have a very prolonged existence.

To turn to another subject, I think it would be advantageous to the profession generally, if engineers were to take some steps towards facilitating the passing of Bills through Parliament. At present, after the formalities of the standing orders have been complied with, and a private Bill has got into the first Committee, which may be of either the Lords or the Commons, every strong point in favour of the Bill is brought forward by the promoters, and every weak point in it is shown by the opponents, and the Committee gives its decision either for or against the Bill proceeding further. If the decision be favourable to the further progress of the Bill, the same formalities have to be gone through again, and the same searching examination has to be repeated in the second Committee, and it is not until this second ordeal has been

successfully passed that the Bill is allowed to proceed and await the Royal Assent. This double examination and delay not only involves excessive cost to the promoters of a private Bill, but, in times of political trouble, may frequently occasion the loss of a Bill without in any way establishing its merits or demerits.

If therefore by some union of engineers (without the intimidation so commonly practised by Trades Unions) a movement could be commenced by which time and money would be saved through the simplification of the forms of the House, not only would the advantages to the profession at large be great, but the country would frequently benefit by the introduction of useful Bills which are kept back through fear of the heavy expenses that would be incurred in bringing them forward.

In conclusion, I would like to add a few words in regard to the progress and prospects of our Institution. It may have been observed by the last Report that the numbers of all classes of members were the largest recorded, but yet the annual increase had not been so great as that of some preceding years. I do not want to assume from this that the summit of our prosperity has been, by any means, reached; but we must not be unmindful of the fact that there is at present a dearth of engineering employment, owing to the depression of trade throughout the world, and that, although this is the parent Institution various other engineering societies have sprung up of late years.

The members of most of these societies are often glad to shelter themselves under the roof and to hold their meetings in the theatre of this Institution, which, as far as it is able, gladly welcomes its children in the parental home. It has long been proposed to enlarge our premises, and to give generally better accommodation in the library and offices, and this would have been taken in hand some years ago had it not been that the promoters of the Westminster (Parliament Street, &c.) Improvement Act had acquired powers to purchase the whole of our property in Great George Street. These powers having now lapsed, the Council will be able to consider the best means of carrying out the idea that prevailed many years ago, viz., that of extending and improving our accommodation to meet the present requirements.

The Address of Mr. Giles having been read, it was moved by Mr. Abernethy, Past-President, seconded by Sir Frederick Bramwell, Past-President, and resolved by acclamation:—

"That a cordial vote of thanks be passed to the President for his Address, and that he be requested to allow it to appear in the Minutes of Proceedings."

The Chairman then distributed the Telford Medals and Premiums, and the Miller Prizes awarded by the Council for the Session 1892–93 (vol. cxiv. pp. 226, 227 and 228.)

21 November, 1893.

SIR BENJAMIN BAKER, K.C.M.G., Vice-President, in the Chair.

(*Paper No. 2730.*)

"The Tansa Works for the Water-Supply of Bombay."

By William John Bird Clerke, B.A., C.I.E., M. Inst. C.E.

The early history of the water-supply of Bombay has been given by the late Mr. H. Conybeare,[1] M. Inst. C.E., in a Paper published in 1858, and by Major Tulloch, R.E. (now Chief Engineering Inspector Local Government Board), in the first chapter of his able report on the water-supply of Bombay,[2] submitted to the Bench of Justices of that city, and published in 1872 when he was their Executive Engineer. In that report, which is in the library of the Institution, Major Tulloch considered exhaustively the various sources from which an additional water-supply for Bombay could be obtained, and in the last chapter, indicated clearly that in his opinion the Tansa project possessed the greatest advantages. Soon after Major Tulloch had left Bombay, the insufficiency of the existing supply from Vehar was impressed upon the authorities, and it was then determined to carry out the Tulsi project, which was one of the smaller ones reported on by Major Tulloch. The works were designed and carried out by Mr. Rienzi Walton, M. Inst. C.E., the Executive Engineer to the Municipality, and were completed in the year 1879, yielding an additional supply of about 5 million gallons per day. The city of Bombay still continued to increase rapidly in size, and in 1884 the supply yielded by Vehar and Tulsi, which amounted to about 16 million gallons per day, was found to be insufficient. The Municipal Corporation of Bombay then determined to consider the Tansa project for the additional supply of the city, and

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[1] "Description of the Works, recently executed for the Water-Supply of Bombay, in the East Indies." Minutes of Proceedings Inst. C.E., vol. xvii. p. 355.

[2] "The Water-Supply of Bombay," London, 1872.

applied to the Government of Bombay, through the then Municipal Commissioner, Mr. (now Sir) E. C. K. Ollivant, for an officer to revise and report on Major Tulloch's Tansa project. The services of the Author, who was then in the Public Works Department of the Bombay Government, were placed at the disposal of the Bombay Municipal Corporation in February, 1885, and in August of the same year he submitted a report with general plans and estimates for the work. In December, the Corporation determined to proceed at once with the execution of the Tansa project, and requisitioned the Bombay Government for the continuance of the Author's services to carry out the work. The contract for the construction of the masonry dam at Tansa was entered into, and work was commenced in January 1886. In November the construction of the aqueduct works from Tansa to Ghatkopar, within 9 miles of Bombay, was begun. The delivery of the cast-iron pipes required for the siphons was commenced in January 1887. The works were completed early in 1892, and were opened on the 31st March in that year, by His Excellency the Marquis of Lansdowne, Viceroy and Governor-General of India.

General Description.

The works consist of a storage lake constructed in the Tansa valley, a distance of 57 miles from Bombay, and an aqueduct consisting of conduits, tunnels, and cast-iron siphons, &c., for bringing the water from the lake for delivery in Bombay. Fig. 1, Plate 1, shows the general position of the storage lake, aqueduct, &c.

The Storage Lake.

The lake is formed by a masonry dam built across the Tansa valley. The catchment-area above the dam, inclusive of the lake surface is 52·5 square miles. Rainfall occurs only during the period of the south-west monsoon, that is, between June and October. The average annual rainfall, as derived from observations taken for the last seven years at the dam site, is about 102 inches. The rainfall returns for this series of years is given in Appendix A. The average rainfall on the whole catchment-area is no doubt higher, but this is not a question of importance, as the capabilities of the Tansa gathering-ground are in excess of any demands which Bombay is ever likely to make on it. The ratio which discharge

bears to rainfall varies greatly with the character of the fall. In a catchment-area such as the Tansa valley, the assumption that at least one-third of the fall is in every year available, appears to be on the safe side. Taking the mean area of the lake surface between full supply and the lowest level to which it can be drawn off, to be 4·6 square miles, the quantity of water available annually would, on the above data, be—

$$47{\cdot}9 \times 640 \times 43560 \times \frac{102}{3 \times 12} = 3{,}784{,}000{,}000 \text{ cubic feet.}$$

$$4{\cdot}6 \times 640 \times 43560 \times \frac{102}{12} = 1{,}094{,}000{,}000 \quad ,,$$

$$\text{Gross quantity available} = \underline{4{,}878{,}000{,}000} \quad ,,$$

which is equal to 30,487,000,000 gallons. From this it is apparent that a rainfall much below the average that has been taken, would afford an ample supply from the catchment-area. The lake, as at present constructed, has its full-supply level or crest of waste-weir at R. L.[1] 405. The sill of the lowest sluices is at R. L. 380. The available storage is that between R. Ls. 385 and 405. The water-spread at R. L. 385 is 3·75 square miles, and at R. L. 405 it is 5·5 square miles. The gross capacity between these two levels is 2,574,000,000 cubic feet; from this has to be deducted the loss due to evaporation and absorption which is taken at 6 feet vertical on the mean area, and is equal to 773,000,000 cubic feet. This makes the available quantity 1,801,000,000 cubic feet or 11,256,000,000 gallons, which is equal to 31,000,000 gallons per day for 365 days; and as the waste-weir runs for at least three months in the year, the actual quantity available would be 41,000,000 gallons per day. This probably is more than Bombay, which had, at the last census, 1891, a population of 821,764, will ever require from Tansa; but, should the time ever arrive when this supply is insufficient, the dam has been so designed that it may be raised to a height which will make the full supply of the lake at R. L. 420. At that level the gross capacity will exceed 5,000,000,000 cubic feet, or double the volume of the lake as now constructed, and the capabilities of the gathering-ground are sufficient for the larger capacity. The land has been acquired to the contour of R. L. 425, *i.e.* the top of the higher dam. The

[1] All levels connected with Municipal operations in Bombay are referred to a datum plane known as "Town Hall Datum," which is 80·14 feet below mean sea-level.

area contained within this contour is 8 square miles, and comprised originally seven villages, of huts formed of a light framework of rough timber plastered with mud, and thatched with grass or straw. The Government Forest Department has acquired the lands surrounding the 425 contour for reserved forests, which form a belt 1 to 2 miles in width. The longitudinal section along the axis of the dam is shown in Fig. 2, Plate 1. On plan, the line of the dam is formed by two straight lines intersecting at an angle of 12½ degrees at the point marked A on the section, the angle of intersection being on the outer side. The dam as now constructed is finished off at R. L. 408, and at this level it has a length of 8,800 feet. Towards the south end, a portion 1,650 feet in length has been kept at R. L. 405 to act as a waste-weir, and the water passes thence by depressions in the ground into the old course of the river. The maximum flood has been taken as that due to a run-off of ¾ inch per hour from the whole catchment-area, which is equal to 25,000 cubic feet per second, or 744 cubic feet per second per 1,000 acres. To discharge this quantity over the waste-weir, 1,650 feet in length, taking the coefficient as 0·7, the lake surface should rise 2·5 feet above the crest of the waste-weir, or to R. L. 407·5. During the eight years in which the Tansa valley has been under observation, no flood of this magnitude has occurred. The largest flood during this period did not exceed 17,000 cubic feet per second. That such a flood as that for which provision has been made, may occur in any season is, however, by no means improbable. The dam, including the waste-weir portion, is of uniform section throughout, that is, for any height above the foundations the section is the upper portion of the general design corresponding to that height. The highest part of the dam is where it crosses the bed of the Tansa river. The foundations here are at R. L. 290, making the maximum height of the dam, as now constructed, 118 feet, and the maximum height to which it may hereafter, if found necessary, be raised, 135 feet. Fig. 3, Plate 1, shows the dam as now constructed, and Fig. 4 shows the design for the dam raised to the full height of 135 feet. In designing the section for the dam, the Author has followed generally the methods advanced by M. Bouvier,[1] which are very clearly explained in Sir Guilford L. Molesworth's note upon high masonry dams. The section as designed, while fulfilling all requirements for stability, is economical as

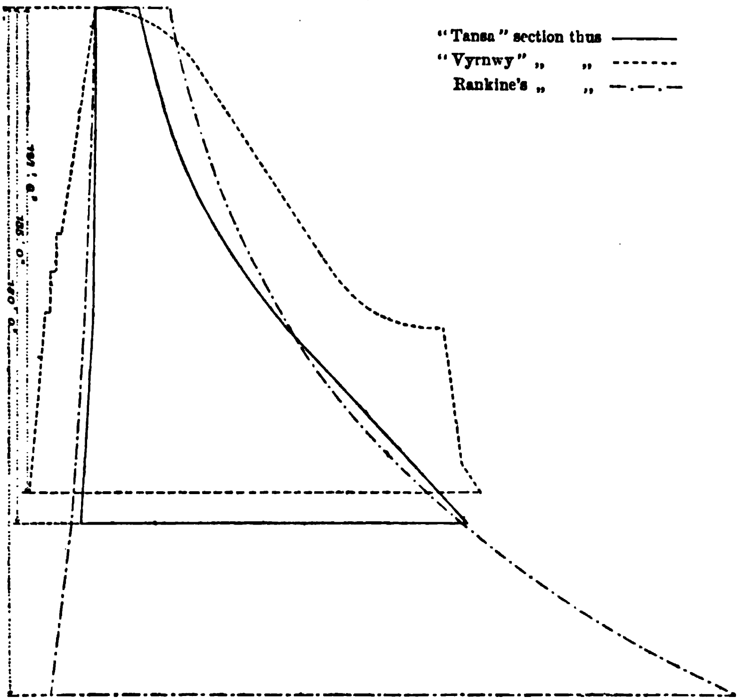
[1] Annales des Ponts et Chaussées, vol. x. 5th Series, 1875, p. 173 *et seq.*

regards material. Comparing it with the section known as Rankine's, the latter has,

For a dam	50	feet high,	an excess of	43	per cent.	of masonry.
„	75	„	„	30	„	„
„	100	„	„	19	„	„
„	135	„	„	9	„	„

Compared with the magnificent Vyrnwy dam of the Liverpool Waterworks, the Tansa dam has considerably less than two-thirds

Fig. 5.



Scale, 1 inch = 50 feet.

of its sectional area. *Fig. 5* shows the comparative sections of the Tansa and Vyrnwy dams and that proposed by Rankine.

The geological formation of the Tansa valley and the whole of the surrounding country is trap. Trial shafts were sunk along the line of dam to determine the depths of the foundations, and the probable rock-surface as inferred from them is shown on the section, Fig. 2, Plate 1. In his original report on the project, the Author, referring to the subject of the trial pits and the line of rock derived therefrom, remarked : " In a trap formation, until the entire

length of foundations has been opened, it is impossible to say how the line of rock may run." The results verified this forecast. In one place the foundations had to be carried 30 feet below the rock level, as ascertained from the trial shafts. This was due to the fact that the rock reached by the trial shafts was, in many cases, a mass of crystalline basalt overlying the bed of amygdaloid trap, which forms the true basin of the valley. This basalt, so
21 far as regards stability, was as good a foundation as could be desired, but it was intersected by veins of soft material (clay, ash, &c.), varying between ½ inch and 2 inches in width, which, under the heavy water-pressure to which they would be subjected, would probably be washed out and form regular passages for the water, thus entailing a serious loss by leakage. Along the whole length of the foundations the aim was, therefore, to pass through the crystalline basalt wherever it occurred, and to lay the foundations of the dam in the amygdaloid trap. This in some cases entailed a heavy outlay, but the result has been satisfactory, as the foundations are quite water-tight. The dam is constructed of uncoursed rubble masonry throughout. Anything approaching regular horizontal joints was carefully avoided, and pains were taken to preserve a good bond throughout the whole breadth of the work. The stones were set in the work as received from the quarries, without further dressing of any sort beyond knocking off weak corners and edges with the hammer. The greater part of the stone used in the work is small, not averaging more than ½ cubic foot. Every stone was laid full in mortar, each one being selected so as to roughly fit the place it was to be laid in; it was then driven home in its bedding of mortar with a light mallet, and all spaces between it and the adjacent stones were filled flush with mortar; spalls or small stones were then inserted in the mortar between the joints. Great care was taken, by a system of close supervision, to prevent, as far as possible, any dry work or hollow spaces being allowed in the masonry. There is no ashlar work in the faces, though the face-stones had to be roughly shaped with the hammer, so as to preserve the outline of the profile. Where it was convenient to do so, large masses of stone were placed in the body of the work; each mass was bedded full in mortar, and built round with rubble masonry. The use of this method of construction was very limited, and it formed an insignificant portion of the bulk of the work.

The stone used is the trap and basalt obtained from numerous quarries opened in the neighbourhood. In quality it varied greatly, and its weight varied between 165 and 185 lbs. per cubic

foot. The inferior qualities, which were usually the lighter, were such as would probably disintegrate by exposure to weather, though quite reliable for use in the heart of a body of masonry. Care was taken to select the best qualities for the exposed surfaces. From the quarries on the down-stream side of the dam tramways were laid to its site, and the material was brought in trucks moved by coolies. There were about 7 miles of tramway laid from the various quarries to the line of the work. From the quarries on the up-stream side of the dam the stone was brought in barges. As the lake grew in size with the raising of the dam, these barges were towed by steam-launches, two of which were employed for the purpose. The whole of the stone used in the work was carried by coolie labour from the place where it was deposited by trucks or barges, on to the work, the bulk of it being in blocks which could be carried by one coolie on the head or shoulder. A small portion of the stone was larger than could be carried by one coolie, and was carried by "now-gunnies," that is, a party of four coolies, who sling the load to be carried from two bamboo poles which rest on their shoulders. As the dam was raised, numerous rough wooden gangways were constructed up the faces for carrying the material.

The lime for the work was obtained from "kunkur," a form of nodular limestone found deposited over the greater part of India. The quantity in the vicinity of the works was limited, and the bulk of it had to be brought from the Nasik districts, above the ghâts, which entailed long carriage by country-cart and railway. The nearest railway-station to the works is Atgaon, on the Great Indian Peninsula Railway, distant 8 miles from the dam. To this station the bulk of the kunkur was brought. A considerable portion of it was carted thence to Tansa, where it was burned on the site of the works. Lime-kilns were also erected at Atgaon station. During the last two seasons in which the work was in progress, when the consumption of material was very great, a portion of the kunkur was burned where it was obtained, above the ghâts, and the lime was brought by rail to Atgaon station. The fuel for burning the kunkur was principally wood obtained from the surrounding forests. In some cases the wood was converted into charcoal before being used in the kilns, and coal ash, obtained from the railway locomotive-yards, was made use of to a limited extent. The quality of the lime did not vary to any appreciable extent with the fuel used. The lime yielded by the kunkur is fairly hydraulic. 3-inch cubes, moulded from the mortars, if allowed to remain in the air for

periods varying from twenty-four to forty-eight hours, continued to set on being immersed in water.

The sand used for mortar was obtained from the beds of the rivers in the neighbourhood of the works. The greater portion of it was brought from the Vaitarna river, distant about 8 miles from the work. It consisted entirely of disintegrated trap-rock. It could not be classed as either very hard or sharp, and would probably not commend itself to engineers accustomed to deal with quartzose sands. The results, however, have been satisfactory. The mortar was composed of 1 part by measure of slaked lime to 1½ part by measure of washed sand. On each bank of the river, close to the line of dam, four pan-mills driven by steam-power were erected, and in these the bulk of the mortar used in the work was mixed. For the flanks of the dam, or positions at some distance from the river, the mortar was mixed in mills known as "ghânis." These consist of a circular trough of masonry 30 to 40 feet in diameter, in which edge-stones some 3 feet in diameter are made to revolve, the motive-power being furnished by bullocks or buffaloes. This process of mixing is slow, but it has the advantage that a cheap mill can be quickly erected close to the spot where the mortar is required. At one time there were about thirty such mills in use; in addition to the two sets of steam-mills which were situated one on each bank of the river.

The results of tests made during the operations showed that the mortar mixed in the steam-pans was somewhat better than that mixed in the "ghânis." The mortar was carried from the ground on to the work by coolie labour. Women were mostly employed on this work. They carried the mortar in iron pans or "ghamelas" on their heads, the quantity carried in each ghamela being about one-third of a cubic foot. As the height of the dam increased, the amount of labour employed in carrying materials was very great. During the last two seasons there were often between 500 and 600 masons employed on the work. In the month of January 1891, the masonry work amounted to 705,000 cubic feet, or 27,000 cubic feet per day. At this period the total number of persons employed in connection with the dam works, including those engaged in the quarries, lime-getting, sand-getting, &c., was 8,000. The total quantity of masonry in the dam is upwards of 11,000,000 cubic feet. The mortar used in the work averaged 32 cubic feet in each 100 cubic feet of masonry. The total quantity of excavation for the foundations was 6,780,000 cubic feet, of which 4,800,000 cubic feet had to be got by blasting. On the water-face of the dam, the joints were

raked out to a depth of 1 inch and filled in with mortar gauged 1 of Portland cement to 1 of fine sand.

There is no flow in the Tansa river after November, and in January 1886, when work was started, the nearest water was more than a mile lower down the stream than the site of the work, where a natural barrier across the river had formed a pond. A steam-pump was erected there and water was pumped up to the works both for use in construction and for the water-supply of the large body of labourers who had come and settled there. In a climate such as that of India, an abundant supply of water is essential to the production of good masonry, and the object aimed at in the first season's operations at Tansa was to get a portion of the dam built across the river-bed so as to impound water above it for the next season's work. It was the middle of March before the foundations in the river-bed were ready for building on. This left only two months and a half to the time when the works would have to be suspended owing to the approach of the monsoon. The dam at this place has a width of 100 feet at the base, and the quantity of masonry in the full section and 20 feet high across the river-bed would have been 600,000 cubic feet. With the resources at command on first starting the works it would not have been possible to build this quantity of masonry in the time at disposal. It was, therefore, decided to build only a portion of the cross-section of the dam, finishing it off according to the drawing on the up-stream face, and racking it off on the down-stream side so as to admit of the completed section being bonded to it in the following season. The section was commenced 40 feet wide at the base, and was racked back at a slope of 1 to 1 on the down-stream side. This dam was raised to a height of 20 feet before operations had to be suspended. The storage thus formed provided an ample supply of water for the next season's work, and, as the storage was increased in each succeeding year, there was no further anxiety as regarded sufficiency of water for all purposes. The water was pumped from the lake by steam-pumps into cisterns erected in elevated positions, and from these it was distributed over the works by a system of piping. From these cisterns, water was also supplied to stand-pipes situated in convenient positions for the use of the work-people who had settled in the vicinity.

The passing of the flood-waters in each monsoon, during the time of construction, was a subject which required consideration. The magnitude of the floods which had to be dealt with did not admit of their being passed, as is the usual practice in this country,

through culverts in the lower part of the dam. For the first three seasons of construction, namely, those of 1886, 1887 and 1888, the floods were allowed to pass directly over the portion of the dam across the river-bed. The length of weir available here was somewhat less than 400 feet, and in heavy floods the water used to pass over it to a depth exceeding 6 feet. The top of the dam was left quite rough, ready for work to be resumed on it, and it is, perhaps, worthy of note that not a stone was at any time displaced on the top of the dam by the action of this large body of water passing over it. In the season of 1888, the height of the dam across the river was 51 feet above the foundations in the river-bed, and in the next season the height would have been 70 to 80 feet.

It was considered inadvisable to subject the foundations to the shock which the fall of so large a body of water through this height would cause, and it was determined not to pass the flood-water over the dam directly into the river after the season of 1888. For passing off the floods in the following year a portion of the dam, about 450 feet in length, on the north bank of the river, was left at R. L. 357. The natural surface of the ground along this length varied between R. L. 356 and R. L. 363, and considerable excavation was necessary to provide a temporary channel for the passage of the flood-water into the bed of the river. A wall was built across the foundation trench from the face of the dam into the natural ground, to prevent the water from passing down directly by the toe of the dam into the river-bed. This acted efficiently for a time, but in a heavy flood the ground became eroded from the end of the cross-wall, and some of the flood-water turned round it and flowed down by the toe of the dam at a high velocity into the river-bed. The Author's deputy, Mr. T. C. H. White, Assoc. M. Inst. C.E., who was on the dam at the time, noticed the sound of some very heavy action taking place at the toe of the dam in the river-bed. After the floods had subsided, it was found that large masses of rock which were lying in the river-bed had been churned about by the action of the water, knocking against the toe of the dam. Some of these masses of rock, which had been very rough, had their surfaces worn quite smooth, and their action on the face of the dam near its toe was clearly perceptible. After the close of the monsoon, excavations were made at the toe of the dam, and the water was pumped out. The dam and the rock foundations on which it rested were carefully examined, and it was found that no injury had been sustained. For passing off the floods of 1890, a portion of the dam 450 feet in length, on the south bank of the river, was left

at R. L. 380. From this a channel was excavated which allowed the flood-water to pass into the river-bed some distance below the point where the dam crosses it. Before the monsoon of 1891 the dam was completed, and the floods of that season passed off by the waste-weir, 1,650 feet in length, provided on the south flank of the work. As previously mentioned, the flood-waters pass from this waste-weir into the original bed of the river by depressions in the natural ground. In finding its way back to the river, the water has excavated several new channels, scouring away all the softer material till it came to the hard rock. Some of these channels now appear as chasms upwards of 40 feet in depth below the surface of the ground.

The considerations which fixed the sill of the lowest outlet-sluice at R. L. 380, were the level at which the water should be capable of being delivered in Bombay and the loss of head in its passage from the lake to the city. The lowest point of the foundation of the dam is at R. L. 290. There is, therefore, in the deepest part of the lake a depth of 90 feet of water unutilized, except for the purpose of keeping the water up to the required outlet-level. The water is drawn off directly by culverts through the dam. There are two outlet-culverts, each 2 feet 3 inches by 2 feet 6 inches, having their sills at R. L. 380. There is one culvert of the same size at R. L. 390, and another at R. L. 400. These culverts are closed by sluice-gates on the inner face of the dam, worked by bevel-gearing from the top. The water in the lake should never fall so low as to expose the lowest two outlets. For the purpose of examining the sluices for repairs, &c., a rectangular masonry chamber has been built into the lake, one of its sides forming the portion of the dam which contains the two outlet-culverts. In the wall of the chamber opposite the dam are two openings corresponding to the two outlet-culverts, but somewhat larger. These openings can be closed by needles or planks let down through grooves in the wall from the top. The top of the wall is at R. L. 395, and when the water in the lake falls to this level the openings can be closed, and the chamber is rendered practically dry for the examination of the sluice-gates.

All the outlet-culverts discharge into an outlet-basin, 30 feet in diameter, on the outer side of the dam, the lowest two culverts directly into the basin, and those at R.-Ls. 390 and 400 on to a flight of ashlar steps which leads down to the basin. On the diameter of the latter parallel to the line of the dam, is a set of four copper-wire screens supported by ashlar pillars. On the

tops of the pillars and on the side-walls of the basin are fixed wrought-iron standards, which carry a rolled beam on top. By differential pulleys attached to this beam the screens are lifted for the purpose of cleaning, &c. The aqueduct leads from the side of the basin opposite to the dam. The arrangements of the outlet-culverts and basin, without the ironwork of the gates and gearing, are shown in Figs. 6, 7 and 8, Plate 2.

Some little time after the outlet-basin was brought into operation an accident occurred, by which the ashlar pillars were overturned. These were of uniform section, 3 feet by 2 feet 6 inches, for their whole height. At the time of the accident the height of the water in the basin below the screens was 4·15 feet. By calculation, the height of water above the screens should have been 5·8 feet to overturn the pillars. If the screens are kept fairly clean the difference of water-level above and below the screens should never be more than a few inches. What the actual height above the screens was at the time of the accident cannot be known. The men in charge stated it was not nearly so much as 5·8 feet, but it must be remembered that any difference of level exceeding a few inches, would have indicated carelessness on their part in attending to the cleaning of the screens. The pillars have been re-built with a greater length of base.

The above account of the Tansa storage lake does not present anything very novel in design or construction, but the conditions of the country, climatic and otherwise, rendered its execution a work of no ordinary difficulty. The situation of the lake is in a thick jungle. When the Author first went there with his surveying party, it was necessary to clear wide avenues through the forest before any survey operations could be carried on for the purpose of fixing the location of the dam. During the monsoon all work had to be suspended, and this reduced the working year to eight months. There is a heavy rainfall, and from the end of the monsoon in October until January or February, malarial fever of a bad type is prevalent. It is a district in which cholera has at times been very virulent. Fortunately, during the progress of the work there was no serious outbreak of this disease on the works, though it raged in neighbouring villages; and on more than one occasion stringent measures had to be taken to prevent communication between the works and the infected districts. The country is sparsely inhabited, mostly by jungle tribes, who are of little use on work, and nearly the whole of the labour had to be imported from other districts. The contractors for these works were Messrs.

Glover & Co., a firm well known in India for more than twenty-five years as contractors for railways, docks, &c. For the first two seasons, 1886 and 1887, Mr. T. C. Glover, the head of the firm, spent part of each season on the work. Mr. Edward Bedford, one of the partners, was there from the beginning to the end of the undertaking, and to his energy, management, and power of organization, the successful completion of the dam in more than one year within the contract time, is in a great measure due. His health suffered severely from continued residence in the unhealthy surroundings of the Tansa valley.

In considering the subject of the available quantity of water in the lake, the loss due to evaporation and absorption has been taken at 6 feet vertical on its mean area. This figure has been arrived at from various experiments made on reservoirs in India, and it represents the total loss for the year. The actual loss at Tansa is much less; for, during some three months, while the waste-weir is running, the loss due to evaporation may be disregarded, and even for some time after the waste-weir has ceased to run there is an inflow which compensates for the evaporation. In the years 1890 and 1891, observations were made to determine the loss in the lake due to evaporation and absorption after all possibility of inflow had ceased. In the former year the water-spread was 2 square miles, and in the latter 3·25 square miles. The only draw-off from the lake was the water pumped up for consumption on the works, and the effect of this was inappreciable, for 0·001 of a foot on the smaller water-spread is equivalent to over 1,000,000 gallons. The results of the observations made are given in the Table below.

Date.	1890.		1891.	
	R. L. of Surface of Lake.	Vertical Loss in one Month.	R. L. of Surface of Lake.	Vertical Loss in one Month.
		Foot.		Foot.
1st January	356·31		380·05	
		0·39		0·44
1st February	355·92		379·61	
		0·63		0·47
1st March	355·29		379·14	
		0·54		0·63
1st April	354·75		378·51	
		0·57		0·64
1st May	354·18		377·87	
		0·81		0·78
1st June	353·37		377·09	
Loss from 1 January to 1 June	..	2·94	..	2·96

During the progress of the work samples of the mortar used were taken daily and moulded into 3-inch cubes in brass moulds. The samples were numbered and stored in the testing-room, and the date of the sample, its composition and the mill in which it was made, were recorded. From time to time afterwards samples were taken at random and tested for resistance to a crushing stress. The tests were made in a small hydraulic press constructed for the purpose. In addition to the tests made on the works, the Author, in his annual visits to England, took with him a few random samples, and submitted them to Messrs. Kirkaldy and Sons, to be tested. Appendix B gives the results of some of the tests, taken indiscriminately from the records. The results do not present a high degree of uniformity. It may be noted that the tests made by Messrs. Kirkaldy gave, almost invariably, higher results than those obtained by testing duplicate specimens at Tansa—a feature that is doubtless attributable to the accuracy of the testing-apparatus and the exactness of the methods employed by that firm. It may be also observed that the tests of samples of the mortars used in the first season's operations gave generally results somewhat superior to those obtained from samples of corresponding age in subsequent seasons. This is apparently due partly to the fact that in the first season the demand for material was small, and greater care was bestowed on its preparation than was found practicable when the work was in full swing and the demand for material was very great, and partly to the fact that for the same reasons the kunkur could not be selected, but, if reasonably good, was brought into use. At the foot of Appendix B are given the average stresses for mortar six to eleven months old, twelve to twenty-three months old, and twenty-four months old and upwards. The weight of the mortar when quite dry as determined by actual weighing is 115 lbs. per cubic foot. Taking the average weight of the stone at 175 lbs. per cubic foot and the proportion of stone to mortar as 68 to 32, the weight of the dry masonry works out to 155·8 lbs. per cubic foot. Under the existing conditions the masonry is never quite dry, and it would therefore be heavier. In designing the profile of the dam, the weight of the masonry was assumed to be 150 lbs. per cubic foot. It may here be noted that the Tansa dam is very dry. There is no actual leakage noticeable anywhere. In the high parts of the dam there is some "sweating" on the face. This is noticeable in the early morning, but as the day advances it disappears, showing that the evaporation is more rapid than the rate at which the "sweating" takes place.

Mr. Bouvier, in the Paper previously referred to, gives the results of some experiments he made on the mortar used in the Ternay dam of the age of thirty months. In the first experiments the cubes of mortar were placed between two plates, the other four sides of the cubes being free. This was the manner in which the Tansa cubes were tested and the results may therefore admit of comparison. Mr. Bouvier obtained 1,423 lbs. on the square inch as the crushing stress, but it appeared to him that this mode of procedure did not produce the same conditions as those which actually obtain in masonry where the mortar, being confined on all sides, is subject to lateral pressure. He tried to produce the latter condition by placing the specimens in blocks of hard wood, surrounding their vertical faces with card-board and compressing them with vertical packings inserted between the wood and the cardboard. In these experiments he obtained a crushing stress of 2,134 lbs. on the square inch, and in one case a stress of 2,542 lbs. on the square inch. The lime used in the mortar at Ternay was that of Theil, an eminently hydraulic lime.

The subject of mortar has been referred to at some length as it is one of great importance to an engineer called upon to construct a large masonry dam in any part of the world. The Author desires to combat an idea which obtains in some quarters, that nothing but Portland or some similar cement should be used in the construction of such works. A Member of the Institution, not devoid of Indian experience, wrote a letter to the Bombay papers in January 1890, when the Tansa dam works were in progress, in which he strongly expressed his opinion that nothing but masonry in Portland cement should be used in the construction of masonry dams, and he also questioned the reliability of the tests of the Tansa mortar. It is hoped that from the information now given, that it may be seen that the use of Portland cement is not essential in the construction of masonry dams, and that the mortar tests to which exception was taken were made under conditions that placed their results above question. Had Portland cement been used in the construction of the Tansa dam, its cost would have been more than doubled, and this would have rendered the execution of the project almost impracticable from financial considerations.

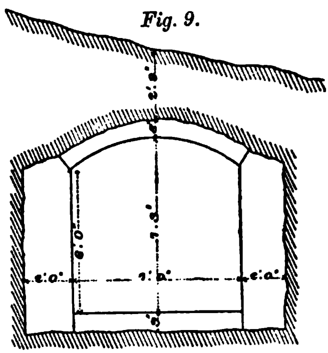
The Aqueduct.

The aqueduct leads from the outlet-basin, and its general position is shown in Fig. 1, Plate 1. The object aimed at in its alignment was to utilize to the fullest extent ground which

was high enough to admit of its being carried in cut-and-cover, tunnels or masonry conduits, and to minimize the use of iron siphons as far as possible. There were two reasons for this:— First, that cut-and-cover, tunnels or masonry aqueducts economized the head, which was a very important consideration, and second, the saving in cost.

The general nature of the country along the line of aqueduct is rugged, and in many places the sidelong slope of the ground is considerable. Seven valleys, of lengths varying between $\frac{3}{4}$ mile and $11\frac{1}{2}$ miles, had to be crossed, and in these the use of iron siphons was obligatory. Beyond Ghât-Kopar, distant $45\frac{1}{8}$ miles from the dam, no high ground is available, and from that point into the City of Bombay iron pipes had to be used. Tunnels were adopted when the saving in length effected thereby was considerable, and in two cases where the precipitous character of the sides of the hills rendered the construction of cut-and-cover almost impracticable. The fall given to the cut-and-cover, tunnels and masonry conduits is 6 inches per mile, and the hydraulic gradient of the siphons is 3·20 feet per mile.

Fig. 9.

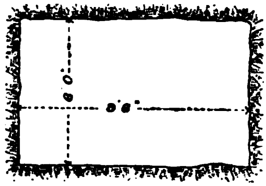


Section of Conduit.
Scale, 1 inch = 8 feet.

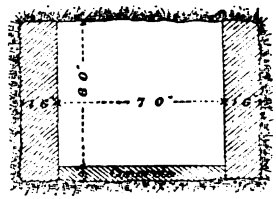
In Appendix C are given the distances and reduced levels of some of the principal points on the line of aqueduct. There are thirteen tunnels, varying in length between 180 and 8,000 feet, the aggregate length being 22,000 feet. The cross-section of the conduit, cut-and-cover, is shown in *Fig. 9.* In his original report the Author calculated that this conduit would discharge 33,000,000 gallons per day. In this calculation he used Bazin's coefficient for rubble masonry. Experiments made since the conduit was brought into operation show that its discharging capacity is considerably in excess of this calculation. Numerous experiments have been made, and the general deduction to be drawn from the results is that the discharging-capacity of the conduit may be fairly arrived at by using Kutter's formula, and taking the value of his coefficient n as 0·012 or 0·013. This gives discharges some 33 per cent. in excess of those arrived at by using Bazin's coefficient for rubble masonry. The cross-section of the tunnels is shown in *Figs. 10.* Where lined, the internal section is the same as that of the conduit. The greater

portion of the tunnels is bored through hard trap-rock which did not require lining, the lined portions being only $\frac{1}{3}$ mile out of the total length of $4\frac{1}{4}$ miles. The internal section of the masonry conduits is the same as that of the cut-and-cover. Fig. 11, Plate 2, shows the design for one of these, a view of which forms the frontispiece of this volume. There are numerous similar structures on the line, differing only in the number of spans, height, &c. The siphons consist of 48-inch cast-iron pipes. This pipe, with a hydraulic gradient of 3·20 feet per mile, would discharge, according to Beardmore's Tables (Eytelwein's formula), 17,000,000 gallons per day. The Author assumed that this would be its

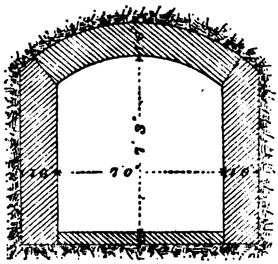
Figs. 10.



Unlined.



Partially lined.



Wholly lined.

TRANSVERSE SECTIONS OF TUNNEL.

Scale, 1 inch = 8 feet.

capacity of discharge, but remarked, in his original report, that it would, when clean, discharge 21,500,000 gallons per day. Experiments made since the works have been brought into operation, have shown that its conveying-capacity while new, exceeds even this figure. The rest of the aqueduct has been designed of double the capacity of the 48-inch siphons; for, when an increase of the supply now given is required, it would be a very serious matter to duplicate them, whereas the siphons can without difficulty be doubled. The walls of the conduit are built of rubble masonry, the lime used being that obtained from the ordinary kunkur of

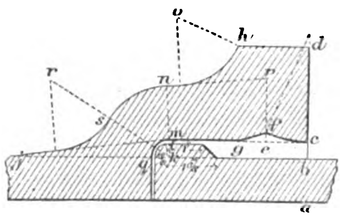
the country. The mortar joints have been raked out to a depth of 1 inch, and filled in with a mixture of one of Portland cement to one of sand. The floor is made of lime concrete; it is plastered with a coating composed of equal parts of Portland cement and sand. The arching consists of hammer-dressed slabs. There are eight manholes to the mile, by which the conduit can be entered for the purpose of examination, cleaning, &c. There are two scouring-sluices in each mile, which can also be used for reducing the water in the conduit should it become overcharged in heavy rains by percolation into the tunnels or from the hill-sides. Observations made since the works have been brought into operation show that, with 17,000,000 gallons per day admitted at the head of the aqueduct, the loss between the head and Ghât-Kopar, a distance of 45⅛ miles, is 175,000 gallons per day, or a little more than 1 per cent. Chambers are provided at the junctions of conduits with siphons. They are rectangular in plan, 14 feet by 10 feet, and are arched in, forming the floor of a wooden cabin erected on the top of each chamber as a residence for the men in charge of the siphon-junction. The conduit enters the chamber at one side, and on the opposite side are two circular openings, in ashlar, to receive 48-inch pipes. One of these openings is for the present built up, as only one pipe has now been laid. The other is closed by a sluice-gate worked by hand from the cabin.

In driving the longer tunnels, rock-drills worked by compressed-air were used; the shorter tunnels were driven by hand-labour. In five out of the thirteen tunnels rock-drills were used.

The levels of the valleys crossed by the siphons vary considerably, giving a maximum head varying between 104 feet in the highest valley and 257 feet in the lowest valley. For siphons in which the maximum head does not exceed 190 feet, pipes 1⅛ inch thick, and for heads in excess of 190 feet 1⅜ inch thick, have been used. The pipes were cast with a working length of 12 feet. The spigot and socket adopted is shown in *Fig. 12.* The calculated weight of each 1⅜-inch pipe is 3·87 tons, and of each 1⅛-inch pipe 3·21 tons. The weight of lead used in each joint was about 100 lbs. The siphons are all laid above ground. Regular tracks formed by excavation and embankment, and provided with under- and over-bridges, culverts, &c., have been made for the siphons. The tracks have been made for two lines of 48-inch pipes (only one of which has now been laid), with a tramway, 2 feet 6 inches gauge, between the two lines of pipes. The tramway was used for carrying the pipes during construction, and it is maintained to facilitate their transport for

repairs. A depot of spare pipes is provided close to every siphon-track. From the ends of the siphon-tracks to the junctions with the conduit in the siphon-chambers, the slope is generally steep. In some places it is almost 2 to 1. On these slopes the pipes are anchored at intervals by heavy blocks of masonry. Special curved pipes of different radii were provided to fit the alterations of gradient on these steep slopes. On the summits of every siphon a double 8-inch air-valve is provided. Between the air-valve and the 48-inch pipe is placed an ordinary sluice-valve, which may be closed when it is necessary to open or take off the air-valve for examination or repair. At every depression along each siphon a 9-inch scour-valve is provided. On siphon No. 6, which is

Fig. 12.



$a\,b$ = $1\frac{3}{8}$ inch ——	$e\,g$ = $\frac{3}{4}$ inch ——
$b\,q$ = $4\frac{3}{4}$ inches ——	$e\,f$ = $\frac{1}{4}$ inch ——
$q\,J$ = 4 inches ——	$f\,P$ = $1\frac{11}{16}$ = $m\,n$
$b\,c$ = $\frac{1}{8}$ inch = $k\,m$	$k\,l$ = $\frac{3}{8}$ inch ——
$c\,d$ = 3 inches ——	$h\,o$ = 2 inches ——
$d\,h$ = $2\frac{1}{4}$ inches ——	$r\,s$ = $k\,s$ ——
$c\,e$ = $1\frac{1}{8}$ inch ——	

Calculated weight taking 450 lbs. as the weight of one C foot of cast-iron :—
12 feet — $4\frac{3}{4}$ inches Pipe, 8,677 lbs.
9 feet — $4\frac{3}{4}$ inches Pipe, 6,677 lbs.

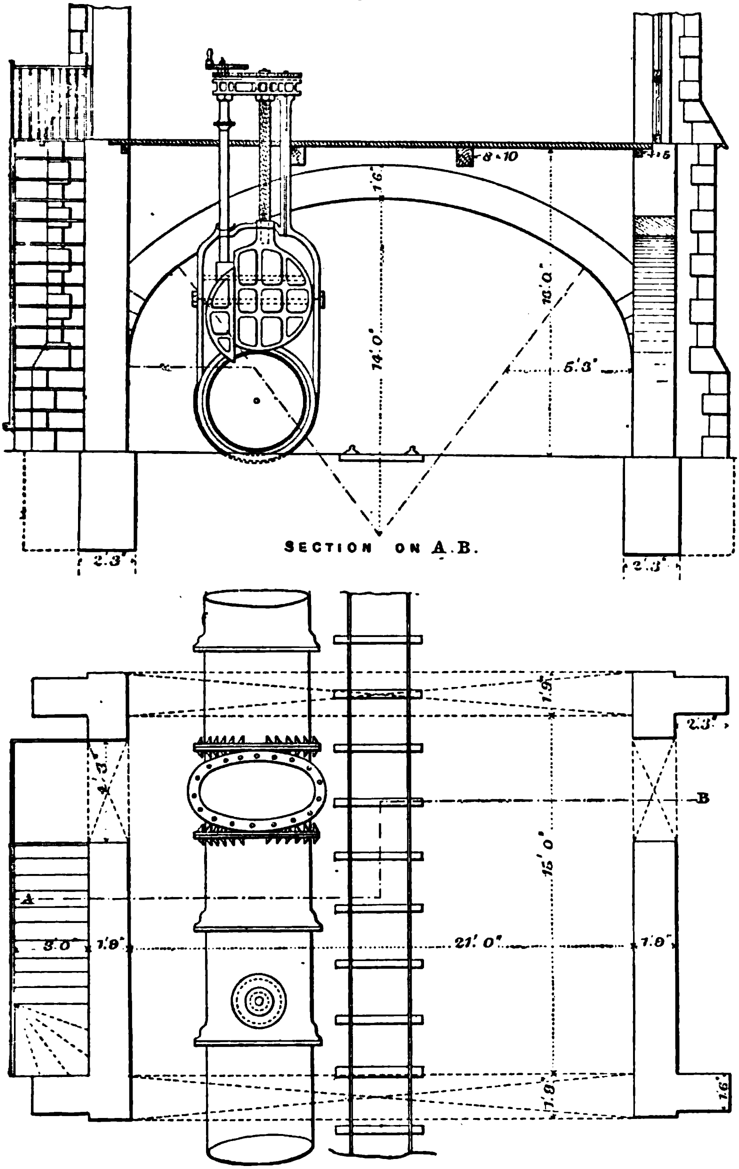
SOCKET-AND-SPIGOT OF 48-INCH PIPE.

Scale $\frac{1}{6}$.

$11\frac{1}{2}$ miles long, four 48-inch stop-valves are provided. These valves are the Glenfield Company's pattern with two shutters. On each stop-valve is a valve-house, from which the valve is worked and which provides accommodation for the attendants. The valve-house is so arranged as not to interfere with the passage of the tramway between the two lines of pipe (*Figs. 13*).

In the 32nd mile from the head, siphon No. 6 crosses the Bassein Creek, a tidal creek separating the island of Salsette from the main-land. At the point selected for crossing, the creek is divided into three channels by low-lying islands. The main channel is 1,500 feet wide, and each of the two smaller channels is 400 feet wide. The pipe is carried across these channels on bridges which are all of one type. The spans are 100 feet between centres. Each

Figs. 13.



Sluice House, Ground Plan.

Scale, 1 inch = 8 feet.

abutment and pier consists of a pair of cast-iron cylinders 5 feet in diameter up to low-water level, where they taper to 4 feet in diameter. From low-water level to the caps, each pair of cylinders is braced together with heavy wrought-iron diagonal and horizontal bracings. The cylinders are all sunk till they reach the rock bed underlying the creek. In some cases they had to be sunk 70 feet below the bed of the creek. Each cylinder is filled with lime concrete for its whole height. The superstructure consists of a pair of wrought-iron lattice-girders, placed 18 feet apart between centres. On the lower flange are placed cross-girders of rolled beams, 9 feet between centres. The line of 48-inch pipe rests directly on the cross-girders. It is placed close to the main girder, and there is a similar space alongside the other main girder for the second line of pipe when it comes to be laid. Along the middle of the bridge, upon the cross-girders, is laid a pair of longitudinal rolled beams, which carry a flooring and the line of tramway. The construction of the aqueduct from Tansa to Ghât Kopar presented many difficulties. The climatic conditions were at least as unfavourable along the line of these works as they were at the dam. The line of country is very rugged, and communication was difficult. There was a great scarcity of water during the greater part of the working season, and in some cases the water required for the work had to be brought from considerable distances. Most of the material for the construction of the conduit had to be carried up the steep sides of the hills from the valleys below either by coolie labour or on donkeys' backs; and the labour, as in the case of dam works, had to be largely imported from other districts. At some periods there were as many as 20,000 people employed along the line of the aqueduct. The contractors for the whole of this work, except the supply of the pipes, were Messrs. Walsh, Lovett, Mitchell & Co., who laboured under considerable disadvantages. They had not previously undertaken any work of this character, and their staff suffered severely from the unhealthy climate. Mr. W. James, Assoc. M. Inst. C.E., who was a partner in the firm, and was in charge of the works, died in February, 1889—his death being attributed to the effects of anxiety and overwork on a constitution impaired by malarious fever. Mr. T. B. Hall, Assoc. M. Inst. C.E., who, after Mr. James's death was in charge of the contract, was obliged to return to England invalided in the early part of 1890. Notwithstanding the difficulties they had to contend against, the contractors resolutely carried out their contract to a successful issue, though, it is believed, at a considerable pecuniary loss.

From Ghât-Kopar siphon-chamber the 48-inch pipe has been continued to Chinchpokli, a distance of $9\frac{1}{3}$ miles. This pipe is laid on a pipe-track as far as is practicable—that is, until it enters the streets of Bombay, where it is laid in trenches. It is provided with air-valves, scour-valves, four stop-valves and valve-houses similar to those described on siphon No. 6. The pipe-track crosses the Great India Peninsula Railway at the Ghât-Kopar station. It is carried over the railway on a plate-girder bridge, which is constructed for the double line of pipes and the tramway. At the 49th mile from the dam it crosses the Kurla Creek, a small tidal creek which separates the island of Bombay from the island of Salsette. The pipe could not be carried over this creek with sufficient headway for the boat traffic which exists on it. It was, therefore, laid under the bed of the creek. Pipes of extra thickness, in lengths of 7 feet 6 inches, with flanged joints, were used here, and are laid on and surrounded with concrete. The continuity of the tramway is provided for by a swing-bridge over the creek. The works from Ghât-Kopar to Chinchpokli were carried out by Messrs. Sitaram Luxmon and Co., a firm of native contractors in Bombay. At Chinchpokli, which is $54\frac{1}{2}$ miles from Tansa, the 48-inch pipe divides into branches leading to the two existing service-reservoirs at Malabar Hill and Bandarwada Hill, and directly into the lower part of the city.

The whole of the pipes for the work, amounting to 48,000 tons, were supplied by Messrs. Macfarlane, Strang & Co., of Glasgow. Each pipe was tested with oil in the manufactory to a pressure of 200 lbs. on the square inch, equivalent to 450 feet head of water. After delivery on the works, the pipes were all again tested to a pressure of 100 lbs. on the square inch. The proportion of pipes rejected on this test was about 2·25 per cent. Some of the rejected pipes were afterwards accepted as short lengths, after having been cut down in a pipe-cutting machine sent out to Bombay for that purpose by the contractors. The behaviour of the pipes, as regards expansion and contraction, laid above ground and exposed to the fierce heat of the Indian sun, was a subject which was not altogether free from anxiety. Some experts were of opinion that on the gradients of the pipe-track, some of which were 1 in 40, the pipes would have a tendency to creep down the gradient, and eventually to draw some of the joints. Upon some of the pipes, which were laid for three years, remaining empty all the time, careful observations made during that period failed to detect the slightest movement. After the works came into operation and the siphons were filled with water, the temperature became more

equable, and any tendency there may have been in that direction was naturally diminished. On siphon No. 6, near the Bassein Creek, there is a curve of 500 feet radius. The hydrostatic head at this point is some 250 feet, and a tendency to an outward movement of the line of pipes was observed, and precautions were taken to counteract it. At the siphon under the Kurla Creek the curves on the approaches as originally laid down were sharp, the hydrostatic head being there about 210 feet. The outward movement here was so marked that it was considered advisable to re-align the approaches with easier curves.

Before the rains of 1891, all the siphons, Nos. 1 to 7, were completed, and it was most desirable that they should be tested under the working-pressure as early as possible. The Tansa lake was filled by the rains of 1891, but, owing to the non-completion of some of the tunnels, water could not be passed down the aqueduct from the lake. During the rains, hill-side streams were turned into the conduits at convenient positions, and the water was stored in them and used to test the siphons in 1891. The results were satisfactory. Leaky points on the lines of siphons, and in some cases at the flange-joints of valves, &c., showed themselves. These were remedied by caulking and packing. On the whole of the siphons only one pipe burst. This was on siphon No. 6, at a point where the hydrostatic head is 257 feet. An examination of the pipe which burst showed that it had been partially cracked before. The burst pipe was replaced by another and the siphon was again tested under the full head satisfactorily. On the line of 48-inch pipes between Ghât-Kopar and Chinchpokli, where the pipes were laid in trenches in the streets, there were several bursts after the works were brought into operation. This portion of the line is nearly horizontal, and is not provided with many air-valves. Whether the bursts were due to accumulated air or whether they were due to the fact that the bottom of the trench is in rock excavation, and that sufficient care may not, in all cases, have been exercised to ensure a cushion of soft material between the pipes and the rough rock bed of the trench, the Author is unable to say, as he had left Bombay and could not examine into each case of failure; but he inclines to the opinion that the failures were mostly due to the insufficiency of air-valves.

Cost of the Work.

In his original report the Author estimated the cost of a project of the general character of that which has now been carried out, at Rs. 123,00,000. The actual cost has been Rs. 149,50,000. The principal causes of the excess on the original estimate were :—

1. Large excess on land-compensation. 2. Increased depth in the foundation of the dam. 3. Adoption of tunnels to a greater extent than was originally provided for (only 2½ miles), the actual length of tunnel being 4¼ miles. 4. Difficulties in the crossing of the Kurla Creek. 5. Excess on establishment-charges, owing to the unhealthy climatic conditions of the district. 6. Advanced rates allowed to the contractors for the aqueduct works during the progress of the contract.

The contract rate for the rubble masonry in the dam was Rs. 27·125 per 100 cubic feet. It was estimated at Rs. 27 per 100 cubic feet. The rates for labour in Bombay are low. The average wages of masons, such as were employed on the dam work was about Rs. 0·80 per day. Two masons could build about 100 cubic feet of rubble masonry per day. The rate for the unlined tunnels was Rs. 43·18 per lineal foot. In the longer tunnels where compressed-air machinery was employed, the contractors lost at this rate. The rate for the cast-iron pipes varied with the place of delivery. Most of the pipes were delivered at Kasheli on the Bassein Creek at the rate of £5 15*s.* 8*d.* per ton. For pipes delivered in Bombay, the rate was £5 16*s.* 7*d.* per ton, and for those delivered at Kurla, it was £5 19*s.* 11*d.* per ton.

Staff Employed on the Works.

On the commencement of the works in 1886, Mr. R. McEwen, who was one of the engineering staff of the Bombay Municipality, was appointed Resident Assistant on the dam works at Tansa. In June 1888, his health broke down, and he had to proceed to England. Mr. E. C. Hawkes, Assoc. M. Inst. C.E., who was appointed to succeed him, died in February 1890. His death was no doubt accelerated by malarial fever contracted in the Tansa valley. The services of Mr. Arthur Hill, Assoc. M. Inst. C.E., of the Bombay Public Works Department, were then lent by the Government of Bombay, and he remained as Resident Assistant on the dam works till their completion in 1891. Mr.

T. C. H. White, Assoc. M. Inst. C.E., was the Author's deputy during the whole period of the construction of the works, and, during the Author's annual visits to England, he was in chief charge. The remainder of the engineering staff were natives of India, the senior assistant being Mr. Nusserwanji D. Bhada, Assoc. M. Inst. C.E.

The Paper is accompanied by 11 drawings and 3 photographs, from which Plates 1 and 2, the *Figs.* in the text and the frontispiece of vol. cxv. have been prepared.

APPENDIXES.

APPENDIX A.—Statement showing the Rainfall in Inches at Tansa Lake, 1887–1893.

Date.	1887.					1888.					1889.				
	June.	July.	August.	September.	October.	June.	July.	August.	September.	October.	June.	July.	August.	September.	October.
1	..	0·00	0·42	0·03	..	..	4·53	2·27	0·22	..	0·00	0·09	0·22	0·50	..
2	..	0·42	0·61	0·12	..	..	2·39	0·79	0·15	..	0·00	0·03	0·27	0·30	..
3	..	0·97	4·42	0·16	..	..	2·21	1·09	..	..	0·04	1·09	0·52	0·10	..
4	0·16	1·83	0·56	0·09	..	..	1·23	0·92	..	..	0·00	0·42	0·10	..	..
5	3·05	2·02	0·49	0·05	..	0·31	0·39	0·42	..	..	0·08	1·20	0·43	..	..
6	1·45	3·11	0·13	1·52	..	0·34	0·31	1·10	..	..	0·26	0·06	1·29	..	..
7	1·32	2·76	0·78	1·20	..	1·85	0·09	0·97	..	..	0·11	1·38	0·79	..	2·15
8	0·12	3·41	0·97	0·15	..	1·27	0·04	0·76	0·02	..	0·26	1·33	1·52	..	..
9	0·00	2·21	2·17	2·10	..	0·00	0·03	0·42	..	..	0·00	2·04	0·79	..	3·00
10	0·06	2·35	2·53	0·90	..	0·06	0·05	1·10	0·03	..	0·22	5·10	0·43	..	..
11	0·87	1·53	1·83	0·85	..	0·00	1·51	1·42	0·98	..	0·21	3·62	2·09	..	..
12	0·26	0·52	0·36	0·02	..	0·00	0·91	0·39	0·22	..	1·42	2·72	2·13	..	..
13	0·32	0·46	0·82	..	..	0·06	1·64	0·32	..	..	2·39	1·39	3·07	0·05	..
14	0·63	2·10	0·41	..	..	0·03	1·71	0·29	..	..	0·47	0·16	0·73	..	..
15	4·60	1·71	0·32	..	..	0·00	3·11	0·21	..	..	0·36	0·73	2·71	0·47	..
16	1·96	0·89	0·29	..	..	0·00	3·13	0·41	..	..	0·07	1·41	0·30	0·56	..
17	0·32	0·32	1·03	..	..	0·00	0·45	0·63	0·40	..	0·00	0·47	0·40	0·36	..
18	0·33	0·51	0·27	..	..	0·00	0·62	1·71	..	..	0·13	4·34	1·09	0·58	..
19	0·11	0·32	0·11	..	..	0·10	1·52	2·73	..	..	0·16	6·79	0·51	..	..
20	0·79	1·02	0·33	..	..	0·47	0·39	1·43	..	..	0·35	1·78	0·13	..	..
21	0·86	0·39	0·51	0·04	..	0·06	0·76	0·87	0·52	..	0·17	0·83	0·37	..	..
22	0·03	1·73	1·49	0·16	..	0·12	1·43	0·42	1·25	..	0·53	0·21	1·76	..	..
23	0·00	4·60	1·52	..	..	0·10	0·21	1·50	..	..	0·36	0·16	1·70	..	..
24	0·59	5·09	0·56	..	..	0·04	0·42	4·93	..	..	1·71	1·03	0·32	0·36	..
25	0·86	3·13	0·19	..	..	0·53	0·13	0·60	..	..	2·59	0·42	0·55	1·06	..
26	1·12	2·15	1·05	..	..	2·18	0·09	0·12	..	..	0·97	0·07	0·53	0·12	..
27	0·51	1·16	0·26	..	..	2·09	0·29	0·29	..	..	1·52	0·03	0·40	0·06	..
28	1·92	0·59	0·47	..	..	1·49	1·21	0·19	..	..	2·39	0·32	0·47	0·88	..
29	1·81	0·96	0·39	..	..	2·91	0·76	0·08	..	..	0·83	1·92	0·05	0·31	..
30	0·40	0·19	0·27	..	..	4·85	2·21	0·11	0·05	..	0·00	0·83	0·40	..	..
31	..	0·14	0·29	..	..	..	1·67	0·17	..	..	..	..	0·10	..	..
Total	24·45	48·59	25·85	7·39	..	18·86	35·44	28·66	3·84	..	17·60	41·97	26·17	5·71	5·15
	106·28					86·80					96·69				

APPENDIX A *continued.*—STATEMENT SHOWING THE RAINFALL

Date.	1890. June.	1890. July.	1890. August.	1890. September.	1890. October.	1891. June.	1891. July.	1891. August.	1891. September.	1891. October.
1	1·33	2·10	3·08	0·01	..	0·00	0·73	1·86	2·08	0·00
2	0·00	1·58	1·22	0·00	..	0·00	0·77	0·61	2·30	0·00
3	0·00	0·73	0·57	0·25	1·55	0·00	1·30	4·42	0·29	0·00
4	0·00	0·15	0·35	0·28	1·64	0·00	4·60	2·14	0·08	0·00
5	0·21	0·40	0·40	0·02	0·16	0·00	1·87	0·33	0·39	0·73
6	1·74	1·67	3·63	0·18	0·06	0·00	0·95	0·61	0·39	0·00
7	0·11	1·81	2·58	0·87	0·09	0·07	0·13	1·10	0·08	0·00
8	0·11	0·24	0·14	0·02	0·01	0·00	0·17	0·37	0·37	0·09
9	0·16	0·10	0·25	0·91	0·08	0·10	0·05	0·31	0·64	0·00
10	0·15	0·89	2·28	0·31	..	0·00	0·00	0·09	0·08	0·00
11	0·18	0·64	0·54	1·28	..	0·00	0·00	0·08	0·60	0·00
12	0·99	0·46	0·33	1·56	..	0·00	0·00	0·17	0·00	..
13	0·21	0·24	0·76	0·83	..	0·00	0·00	0·10	0·15	..
14	0·51	0·81	0·86	0·99	..	0·00	0·69	0·04	0·03	..
15	0·08	2·63	1·13	0·15	..	0·00	0·09	0·04	0·03	..
16	0·05	1·12	0·44	0·00	..	0·00	0·92	0·20	0·11	..
17	1·10	0·99	0·17	0·00	..	0·00	2·42	0·62	0·68	..
18	5·01	0·70	0·72	0·00	..	0·00	1·56	0·27	0·24	..
19	4·96	1·20	0·55	2·68	..	0·00	1·19	0·44	0·61	..
20	4·18	3·26	0·82	0·73	..	0·00	0·63	0·22	0·13	..
21	0·64	3·12	0·88	0·46	..	0·00	0·25	0·40	0·00	..
22	0·18	0·78	0·52	0·00	..	0·35	0·86	0·03	0·00	..
23	1·05	1·16	0·07	0·00	..	0·05	0·87	0·00	0·00	..
24	0·85	1·62	0·10	0·08	..	0·00	2·24	0·00	0·13	..
25	0·12	0·97	0·10	0·00	..	0·04	5·33	0·68	1·65	..
26	0·03	0·28	0·00	0·97	..	0·06	4·59	0·42	1·40	..
27	0·11	0·25	0·17	0·30	..	0·37	5·83	2·16	1·22	..
28	0·10	1·66	0·17	..	..	0·14	1·26	1·95	0·86	..
29	0·95	2·15	0·13	..	..	0·12	1·55	0·67	0·58	..
30	1·54	0·15	0·44	..	..	1·35	0·65	0·51	0·00	..
31	..	3·70	0·36	..	..	..	0·91	0·82	..	..
Total	26·65	37·56	23·76	12·88	3·59	2·65	42·41	21·66	15·12	0·82
	104·44					82·66				

IN INCHES AT TANSA LAKE, 1887–1893.

Date.	1892. June.	July.	August.	September.	October.	1893. May.	June.	July.	August.	September.	October.
1	..	0·17	1·93	5·31	0·13	..	..	0·14	1·93	0·70	0·13
2	..	0·16	0·17	3·19	0·03	..	..	0·00	0·84	1·07	0·00
3	1·13	0·82	0·54	3·96	..	..	..	1·06	3·82	0·43	..
4	..	..	0·27	3·78	..	..	..	2·45	3·47	0·43	..
5	0·27	1·02	0·59	2·14	..	..	..	2·60	1·26	0·42	..
6	0·31	0·15	1·01	0·84	..	..	..	4·00	4·45	0·50	..
7	0·89	0·46	1·66	0·30	..	..	..	1·44	1·83	0·31	..
8	0·25	3·72	0·10	0·28	..	..	..	0·66	1·84	0·45	..
9	0·39	3·49	0·37	0·00	..	..	..	0·37	0·77	0·43	..
10	0·14	6·80	2·20	0·25	..	..	0·24	0·16	0·50	0·11	..
11	..	2·76	1·19	0·63	..	..	0·35	0·40	0·40	0·09	0·00
12	0·01	1·89	0·39	0·51	..	..	0·00	0·47	0·09	1·11	0·39
13	0·05	1·15	0·58	0·35	..	..	..	0·60	0·00	0·21	0·00
14	0·31	1·78	1·65	0·08	..	..	0·08	0·34	0·23	0·71	..
15	0·10	1·19	0·25	0·07	..	..	0·14	0·25	0·94	1·59	..
16	1·40	1·13	2·01	1·95	..	..	0·28	1·92	0·99	1·66	..
17	1·24	2·24	2·05	0·13	..	..	3·68	1·91	0·04	0·58	..
18	..	1·84	0·03	0·90	..	..	8·53	0·60	0·00	0·07	..
19	..	0·36	0·29	0·16	..	..	7·09	0·40	0·00	0·02	..
20	0·03	0·21	0·43	0·12	0·03	..	5·12	4·29	0·17	0·56	..
21	0·12	0·41	0·75	0·89	0·10	..	6·22	0·61	0·78	0·02	..
22	0·76	0·54	0·00	0·61	0·21	..	2·24	0·00	0·58	0·00	..
23	0·20	0·23	..	0·14	0·35	..	0·95	0·11	0·85	1·15	..
24	0·24	0·77	0·10	0·34	0·18	0·02	0·06	0·32	0·62	0·03	..
25	..	0·76	0·38	0·65	..	0·00	0·27	0·12	0·61	0·08	..
26	0·04	0·57	3·68	0·03	..	..	0·46	0·33	0·33	0·50	..
27	0·13	1·05	4·76	..	..	2·41	0·14	0·67	1·54	0·00	..
28	0·00	1·47	4·73	..	0·81	2·32	0·16	0·38	1·00	0·04	..
29	0·04	0·70	0·67	..	..	0·85	0·94	0·50	0·53	0·00	..
30	..	0·54	1·20	..	0·21	0·60	0·11	0·82	0·57	0·50	..
31	..	1·23	3·74	..	0·08	0·00	..	0·13	0·76	..	..
	8·05	39·61	37·72	27·61	2·13	6·20	37·06	28·05	31·74	13·77	0·52
			115·12						117·34		

APPENDIX B.

Results of Tests Made on Samples of Mortar used in the Tansa Dam.

Date of Test.	Age of Sample in Months.	Crushing Stress. Lbs. per Sq. Inch.
18/ 3/87	12	945
24/ 5/87	14	1,090
10/10/90	55	2,305 [1]
24/ 5/87	13	1,400
10/10/90	54	2,147 [1]
6/11/87	18	871
7/ 6/88	26	1,058
27/ 9/88	29	1,763 [1]
6/ 4/93	83	2,239 [1]
7/ 6/88	25	1,250
27/ 9/88	29	1,744 [1]
18/ 3/87	10	846
16/11/87	18	1,233
6/ 4/93	83	2,366 [1]
18/ 3/87	10	1,132
27/ 9/88	29	1,877 [1]
30/12/86	7	750
7/ 6/88	25	1,200
27/ 9/88	28	1,824 [1]
30/ 9/87	17	1,120 [1]
6/11/87	18	995
26/11/87	19	1,095
5/ 2/87	9	764
24/ 5/87	12	1,244
7/ 6/87	12	1,320
30/ 9/87	16	1,744 [1]
21/ 5/87	11	1,145
10/ 6/90	48	1,534
10/10/90	52	1,493 [1]
22/ 4/88	22	1,300

Date of Test.	Age of Sample in Months.	Crushing Stress. Lbs. per Sq. Inch.
27/11/87	11	1,020
30/ 4/89	28	1,540
27/11/87	10	647
16/12/88	25	1,269
9/ 4/88	16	870
30/ 4/89	27	1,316
4/ 1/88	10	622
10/10/90	43	1,574 [1]
30/ 9/87	6	681 [1]
6/11/87	8	733
25/ 5/88	13	950
6/ 4/93	72	1,752 [1]
10/ 6/90	37	1,092
10/10/90	41	1,384 [1]
9/11/87	6	600
8/ 6/88	13	900
27/ 9/88	17	1,267 [1]
11/ 5/89	24	1,540
11/ 3/90	34	1,680
13/ 5/88	12	933
14/11/90	42	1,161
4/ 1/88	8	670
8/ 6/88	13	930
27/ 9/88	16	1,030
7/ 6/88	12	520
8/ 6/88	12	622
30/ 4/89	23	1,176
6/ 4/93	70	1,377

Date of Test.	Age of Sample in Months.	Crushing Stress. Lbs. per Sq. Inch.
10/ 6/90	36	1,534
10/10/90	40	1,679 [1]
4/ 1/88	2	385
9/ 4/88	6	600
23/ 9/88	11	1,020
8/ 6/88	7	746
27/ 9/88	11	1,037 [1]
22/ 4/88	6	620
8/ 6/88	7	750
27/ 9/88	11	1,042 [1]
6/ 4/93	64	1,377 [1]
8/ 6/88	7	522
27/ 9/88	10	919 [1]
15/12/88	12	498
24/10/89	22	1,014 [1]
6/ 4/93	63	1,327 [1]
16/12/88	11	522
30/ 4/89	15	1,036
10/ 6/90	28	754
10/10/90	32	1,246 [1]
20/ 4/89	12	982
18/10/88	4	522
24/10/89	16	856
10/ 4/89	6	742
10/ 4/89	6	868
24/10/89	12	1,070 [1]

[1] Test made by Messrs. Kirkaldy & Sons.

APPENDIX B.—RESULTS OF TESTS MADE ON SAMPLES OF MORTAR USED IN THE TANSA DAM—*continued.*

Date of Test.	Age of Sample in Months.	Crushing Stress. Lbs. per Sq. Inch.	Date of Test.	Age of Sample in Months.	Crushing Stress. Lbs. per Sq. Inch.	Date of Test.	Age of Sample in Months.	Crushing Stress. Lbs. per Sq. Inch.
29/12/88	2	199	24/12/89	10	1,232	15/ 4/90	5	468
10/ 4/89	5	588						
24/10/89	12	745 [1]	31/ 5/90	15	1,040	6/ 4/93	40	935 [1,2]
24/12/89	14	1,008						
			24/12/89	9	952	8/ 2/91	14	675
24/12/89	13	728 [2]						
			24/12/89	9	728 [2]	31/12/90	12	513 [2]
10/ 4/89	3	266						
24/12/89	11	924	24/12/89	9	924	9/10/90	9	486
24/12/89	11	1,120	24/12/89	8	728 [2]	6/ 4/93	37	1,392 [1]
10/ 4/89	3	420	6/ 4/93	47	1,722 [1]	18/12/90	9	500
24/12/89	11	929						
			20/12/89	7	840	6/ 4/93	28	723 [1]
24/12/89	11	476						
27/ 1/90	12	680 [2]						
6/ 4/93	50	1,038 [1]	20/12/89	6	784	6/ 4/93	26	588 [1,2]

[1] Test made by Messrs. Kirkaldy & Sons. [2] Mixed in Ghani.

AVERAGE STRESSES OF ALL TESTS.

	Lbs. per Sq. Inch
Samples from 6 to 11 months old	774
„ „ 12 „ 23 „ „	982
„ 24 months old and upwards	1,461

AVERAGE STRESSES OF TESTS MADE BY MESSRS. KIRKALDY & SONS.

	Lbs. per Sq. Inch.
Samples from 6 to 11 months old	920
„ „ 12 to 23 „	1,160
„ 24 months old and upwards	1,551

APPENDIX C.—TABLE OF DISTANCES AND LEVELS OF SOME PRINCIPAL POINTS ON THE AQUEDUCT FROM TANSA TO GHÂT KOPAR, 45⅛ MILES.

Description.	Distance from Outlet-Well at Tansa.	Reduced Level.	Lengths of Conduit and Tunnel portions.	Lengths of Siphon portions.	Fall in Conduit and Tunnel portions.	Fall in Siphon portions.
	Miles-feet.		Miles-feet.	Miles-feet.	Feet.	Feet.
Floor of conduit at outlet-well	..	380·00				
			7–3554	..	5·43	
Siphon. No. 1, north	7–3554	374·57				
				0–3524	..	2·37
No. 1, south	8–1798	372·20				
			2–623	..	1·14	
No. 2, north	10–2421	371·06				
				1–2742	..	5·24
No. 2, south	11–5163	365·82				
			3–1357	..	1·54	
No. 3, north	15–1240	364·28				
				1–1930	..	4·57
No. 3, south	16–3172	359·71				
			1–5028	..	0·97	
No. 4, north	18–2918	358·74				
				0–4526	..	2·95
No. 4, south	19–2164	355·79				
			1–2968	..	0·91	
No. 5, north	20–5132	354·88				
				1–823	..	3·90
No. 5, south	22–675	350·98				
			2–2268	..	1·22	
No. 6, north	24–2943	349·76				
				11–2435	..	36·45
No. 6, south	36–98	313·31				
			6–847	..	3·27	
No. 7, north	42–945	310·04				
				0–4673	..	2·91
No. 7, south	43–338	307·13				
			2–526	..	1·24	
Ghât Kopar siphon-chamber	45–864	305·89				
Totals	..	..	27–1331	17–4813	15·72	58·39

There is a drop of 1·5 feet at the site of the proposed filter-beds at Vândra in the 6th mile.

NOTE.—The levels refer to the lower sides of the pipes at the north and south ends respectively of the several siphons.

(*Paper No. 2684.*)

" The Baroda Waterworks."

By Jagannath Sadasewjee, Assoc. M. Inst. C.E.

(*Abridged.*)

The city of Baroda, containing a population of about one hundred and twenty thousand, was until recently dependent for its water-supply upon tanks and wells in its immediate vicinity. The idea of bringing a new supply of water from the Nerbudda river was first contemplated in 1866 by His Highness the late Maharaja Khanderao; and his proposal was then investigated by the late Mr. A. W. Forde, M. Inst. C.E. The subject subsequently engaged the attention of numerous engineers, and other schemes were projected or investigated by Messrs. T. P. S. Crosthwait, J. H. E. Hart and T. D. Little, MM. Inst. C.E.

Shortly after the accession of the present Maharaja Gaikwar of Baroda, the Author proposed a scheme by which that water could be obtained from the river Surya at Jafferpur, which is within His Highness's territory. The survey of this project was undertaken by the Author in 1883, and, after having been favourably reported upon by Mr. Playford Reynolds, M. Inst. C.E., and by Lieutenant-General C. J. Merriman, C.S.I., R.E., the scheme was sanctioned by the Gaikwar on the 26th of November, 1884, and work was commenced in January 1885.

The undertaking involved the construction of an impounding-reservoir 12 miles north-east of Baroda, with waste-weir and outlet-works; a 30-inch cast-iron main; settling-tanks and purification-works; a covered service-reservoir, and distribution-works. The works were designed to afford a constant daily supply of 3,000,000 gallons to 120,000 consumers.

Impounding-Reservoir.

The catchment-area of the Sayaji Sarovar, as the impounding-reservoir has been named, is 36·2 square miles (Fig. 1, Plate 3). The water-spread at top-water level is 4·72 square miles. The result of 17 years' rainfall observations at Baroda showed a mean rainfall of 39 inches, and a mean rainfall of 33 inches for three con-

secutive dry years.[1] These figures were adopted as applicable to the neighbourhood of the proposed reservoir, being less than those of rainfall observations for the same period at Halol, which lies eastward, as Baroda is westward, of the catchment-area. The loss by evaporation from the reservoir was assumed to be 72 inches annually.

The reservoir is formed by an earthen embankment across the Surya river, 14,400 feet in length, and 54 feet in maximum height, storing 1,287,000,000 cubic feet of water. The embankment is constructed principally of earth excavated from the waste-water-course which is situated at its northern extremity. The top-breadth, which is uniform throughout, is 15 feet. The top of the embankment is 16 feet above the top-water level, or, reduced to mean sea-level, is at R.L. 224·00. The sill of the outlet-valve is at R.L. 188·00. The side slopes above R.L. 217·00 are 1½ to 1; the inner slope below that level is 3 to 1; the outer slope is 2½ to 1 between the Wagli nullah and the Surya river, and 2 to 1 from the former point to the waste-weir, and from the latter point to the southern extremity of the embankment. Before commencing to build, the site throughout was completely cleared of vegetable mould, trees and shrubs, and the sand and silt were removed from the area forming the foundation in the bed of the Surya river and the Wagli nullah.

There is no puddle-wall in the embankment, for the material employed in its construction was of good quality, but one is carried through the natural ground beneath it along its entire length. The bottom width of this wall varies between 10 feet and 15 feet, with side-slopes from ⅛ to 1 to ¼ to 1, according to the nature of the ground; the depth varies between 18 feet and 25 feet—a clay bottom being reached, with the exception of a portion 200 feet in length between the Surya river and the Wagli nullah, at A, Fig. 2, Plate 3. The trench was here excavated 25 feet to 40 feet in depth, and 5 feet wide in the deepest part, about 50 feet in length. A section of the embankment at this place is given in *Fig. 3*. In excavating the trench between the Wagli nullah and the Surya river, springs were met with, and in the middle portion a large quantity of water percolated through the sandy strata. The method followed in filling the trench with puddle was:—A layer of 6 inches to 12 inches of clay was thrown into the trench, water was added, and the clay was trodden by

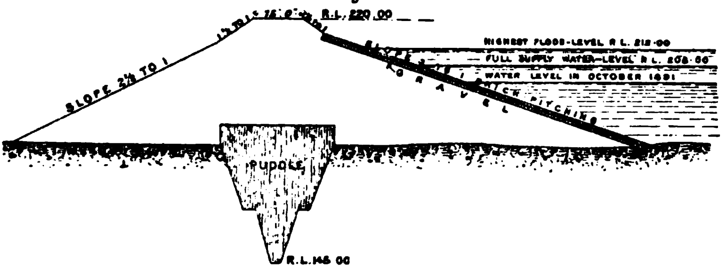
[1] The rainfall of the years 1869, 1870 and 1871 was 37 inches, 34 inches and 28 inches respectively.

labourers until it was well mixed. Another layer was then added and similarly treated until it became thoroughly incorporated with the lower one. This process was repeated until the puddle-wall attained a height of 2 feet above the surface of the ground, so as to form a water-tight junction with the earth-work of the dam.

After the monsoon of 1888, a second puddle-trench was excavated parallel to and about 150 feet distant from the inner toe of the embankment (Fig. 2). The length of this trench was 2,000 feet, and, with the aid of two pulsometers, and one 8-inch and one 4-inch centrifugal pump, driven by portable steam-engines, it was sunk until a good clay bottom was reached. Part of the trench was filled in before the monsoon of 1889 set in, and the remainder was filled in before the commencement of the monsoon of 1890. The object of making the second puddle-wall was to prevent underground percolation, and the Author believes that it has been of material service in this respect.

A gap about 400 feet in length was left in the embankment to

Fig. 3.



permit the passage of flood-water, and was filled up in the early part of 1890. The entire work was raised to R.L. 217·00 before the monsoon of 1890, and was completed to R.L. 224·00 in the beginning of 1891. The portion of the embankment built across the gap sank about 12 inches after the monsoon of 1891. It was subsequently restored to the true level, and stones were fixed at intervals along the whole length of the embankment to aid in observing whether any further settlement took place. The inner slope was hand-pitched with dry bricks 9 inches thick overlying 10 inches of gravel, the joints being well packed with gravel. The pitching has stood well during the monsoons of 1891 and 1892.

During the monsoon of 1890, water was observed to issue from the toe of the outer slope of the embankment near the point A, Fig. 2. This caused some anxiety and apprehension, and Mr. J. E. Whiting, M. Inst. C.E., was consulted with reference to it. He,

however, did not see any cause to consider the embankment unsafe, for reasons given in a report dated the 22nd May, 1891. He advised that the outer toe of the work should be underpinned with a clay wall 12 feet wide, to prevent any tendency to slip which might be anticipated under the circumstances—the embankment having been founded upon black soil, 5 feet to 6 feet deep, for about 4,000 feet of its length. Also that a system of surface-drains should be cut, to lead water falling on the outer face of the embankment into a new drain or cutting which had been already made parallel to the outer toe, and 200 feet to 300 feet distant from the axis of the work. This cutting, which was 10 feet to 12 feet in depth, and 2,900 feet long, discharged upwards of 49,000 cubic feet and 75,000 cubic feet per day, in the months of October 1891 and 1892 respectively. This large quantity of water rose to the surface of the ground at the toe of the outer slope of the embankment from the Wagli nullah to the Surya river; but the excavation of the new cutting between the nullah and the river has rendered the ground near the toe of the outer slope dry, and the Author therefore does not now fear the occurrence of any slips in the black soil there. The sides of the cutting stand well with slopes of 1 to 1 except where the sub-soil is composed of black soil. Here slips do occur.

Waste-Weir.

The waste-weir, Figs. 4, Plate 3, is constructed at a distance of about 1,000 feet from the commencement of the waste-water-course, and is 800 feet in length and 100 feet in width. It is built of two parallel concrete walls, 6 feet wide and 10 feet deep, carried to a solid foundation throughout, and the intermediate space of 88 feet is paved with concrete 1 foot 6 inches in depth. The superstructure on the concrete walls is brickwork in lime-mortar.

The waste-water-course, Fig. 2, has a clear width of 800 feet, and is excavated through the elevated ground which forms the northern boundary of the reservoir. The bed of the channel is level as far as about 3,000 feet from the commencement, and then falls at an inclination of 1 in 150 towards a nullah between Jafarpur and Rasulabad, which discharges into the Surya river. The height of the embankment above the crest of the weir is in the middle portion 16 feet, gradually decreasing to 12 feet at the ends. The ground through which the waste-water-course is excavated consists of clay of varying quality, the bed of the channel being hard clay. It is calculated that the maximum

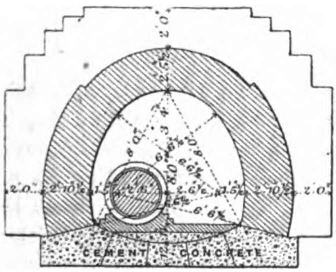
flood-discharge of 11,680 cubic feet per second would rise to a height of about 6 feet above the west end of the waste-water-course. This discharge is equivalent to ½ inch rain per hour off the catchment-area, which is generally flat, and consists largely of grass land.

The Author, having found from the observations that the discharge of the waste-water-course is not more than 530 cubic feet per second between R.L. 209·00, and R.L. 210·00, proposes to alter the level portion from the apron of the waste-weir, 2,000 feet in length, so as to give a slope of 1 in 1,000, and it will then, according to the Author's calculation, discharge 3,370 cubic feet per second.

OUTLET-CULVERT AND VALVE-TOWER. (Fig. 5, Plate 3.)

The draw-off and scour-valves are contained in a tower which is connected with the lake by two plug-valves placed at R.L.

Fig. 6.



SECTION OF OUTLET-CULVERT.

188·00 and R.L. 198·00 respectively, and with a scour-valve, the centre of which is at R.L. 184·25. From the interior of the tower the water is passed into a cast-iron main 30 inches in diameter controlled by a sluice-valve.

A culvert under the embankment, *Fig. 6*, connects the tower with a valve-house at the foot of the outer slope. The arrangement of valves at this point permits the water to be discharged from the reservoir into the Suyra river by an open scour-drain. This provision is for the protection of the plug-valve at R.L. 188·00, which may become choked with silt, as actually occurred during 1892.

AQUEDUCT. (Fig. 7, Plate 3.)

From the valve-house at the foot of the embankment a cast-iron main, 30 inches in diameter, is laid parallel to the road from Ajwa

to Baroda. The trench for a distance of about 2¼ miles from the valve-house has slopes of 1 to 1, and is excavated to a maximum depth of 26 feet. The thickness of the pipes is ¾ inch, calculated for 100 feet head of water.

The main approaches Baroda on the south side of the Ajwa road, and, passing over the Raja tank on brick pillars and through the Pani Gate, is carried to the centre of the Mandvi Tower in the city. Here the pipes are reduced from 30 inches to 24 inches in diameter. The length of the main from the valve-tower to the Mandvi Tower is 12½ miles. The total weight of metal in it is about 7,700 tons and the number of pipes is about 5,670, exclusive of bends and branches. It is provided with 12 stop-valves and 12 ball air-valves, at regular intervals of 1 mile, and with 6 scour-valves placed in convenient situations for supplying water to certain villages in times of scarcity. The distributing-pipes laid throughout the city vary between 16 inches and 3 inches in diameter, and house-connections are generally made with ½-inch or ¾-inch ferrules.

Purification Works.

At Nimeta, 5 miles from the reservoir, there are 3 Anderson revolving water-purifiers, 2 settling-tanks, 3 filter-beds and 1 covered service-reservoir. The revolving purifiers are driven by a steam-engine, and make one revolution in a minute-and-a-half. The cylinders are daily supplied with 20 lbs. of iron borings. The water, after having been in contact with the iron borings, is led from the cylinders to the settling-tanks, which worked alternately. Appendix, Table I, shows clearly that the water from the Sayaji reservoir is slightly improved after passing through the Anderson purifier into the settling-tank at Nimeta, and is further improved by passing through the filters into the covered service-reservoir. The analysis of the filtered water from the stand-posts in the city (Table II) shows that the water deteriorates slightly after leaving Nimeta. The Author has introduced a Root blower for supplying air to the water in the tank connected with the Anderson purifiers, so as to thoroughly aërate the water before its passage into the settling-tanks. A similar mode of aërating water had been successfully practised at Agra, and the result obtained at Baroda is very satisfactory (Table III).

The settling tanks are each 404 feet by 394 feet, and 13 feet deep, built of brickwork in lime mortar. The filters are 160 feet by 100 feet by 13 feet. The walls are of brickwork, and the floor is formed of concrete 2 feet thick, with collecting-drains and main

drain arranged herring-bone fashion. A layer of dry bricks with 4½-inch passages between adjacent rows, covered by a flooring of bricks on edge, supports 1 foot 6 inches of stone broken to pass through a 2-inch ring. The filtering medium consists of a layer of fine sand 2 feet in thickness, brought from the Oorsung river. The rate of filtration is 4 gallons per square foot of filter per hour.

Service-Reservoir.

The walls of this reservoir are constructed of brickwork in lime mortar, and measure 328 feet in length and 121 feet 6 inches in breadth. The roof is carried by 184 brick pillars, 14 feet apart from centre to centre, built with Portland cement. The floor is of concrete 2 feet thick plastered with lime. The roof is of concrete carried by steel girders and joists. The principal girders, which are 10 inches × 5 inches × ⅝ inch, are 14 feet apart from centre to centre; on these the cross girders, 7 inches × 3¾ inches × ½ inch, are placed, 4 feet 8 inches apart from centre to centre, and between the cross girders steel joists, 2 inches × 1½ inch × ¼ inch and 4 feet 8 inches long, placed 1 foot apart, carry the concrete covering, which is 7 inches to 8 inches thick, and is covered by a foot of earth. There are 16 cast-iron ventilators 10 inches in diameter in the roof, and 14 openings, 2 feet by 3 feet, for the admission of light when required. The depth of water in the reservoir is 16 feet, an 18-inch pipe being provided to carry any overflow into a scour-drain which serves the service-reservoir, settling-tanks and filter-beds. The works were completed in March, 1892, the total cost being Rs.34,40,000.

Conclusion.

The results of gauging the Surya river for five years from 1885 to 1889 has not been very satisfactory, as may be seen from the following Table :—

Year.	Rainfall.	Discharge.	Rainfall Discharged.
	Inches.	Million Cubic Feet.	Per cent.
1885	19·81	359	21
1886	42·47	1,076	30
1887	35·85	406	13
1888	19·97	123	17
1889	45·79	523	13

During this period, the year 1886 afforded comparatively good results, but the years 1887 and 1889 gave a scanty yield of water.

It is intended to supplement the works by throwing an earthen dam across the Vishwamitri river to form a reservoir at the village of Asoj, and to cut a canal about 4 miles in length to feed the Sayaji Sarovar. This will impound about 625 million cubic feet of water. In 1893 the Sayaji Sarovar rose to R.L. 210·10, that is, 2 feet above the full-supply level (R.L. 208·00). In view, however, of the recurrence of dry seasons, it is desirable that the present works should be supplemented as suggested.

The Author desires to express his indebtedness to Rao Saheb Hurrichandra Gopal, Executive Engineer, to Mr. Philipps, the Mechanical Engineer, and to Mr. Ramchandra Harry, Sub-Engineer, as well as to all others who co-operated with him in carrying out this work to its successful issue. Especially he desires to acknowledge the great obligation conferred on him by Mr. Playford Reynolds, late Chief Engineer of Baroda State, for the valuable advice that he gave from time to time during the execution of the works. The present Chief Engineer, Mr. Graham R. Lynn, M. Inst. C.E., has also given the Author the advantage of his hearty support in the completion of the undertaking, which was formally opened by the Gaikwar on the 29th of March, 1892.

The Paper is accompanied by 12 plans, from a selection of which Plate 3 and the *Figs.* in the text have been prepared; and by 10 Appendixes containing copies of Reports on the works, and rainfall and other statistics, from which some of the information presented in the Paper has been extracted.

APPENDIX.

TABLE I.—ANALYSIS OF WATER COLLECTED ON THE 11TH APRIL, 1892, BY DR. D. R. DHUME, L. M. & S. CHEMICAL ANALYST, BARODA STATE.

No.	Water Collected from different Localities.	Grains per Gallon. Total Solids. (a) Before incineration.	Grains per Gallon. Total Solids. (b) After incineration.	Grains per Gallon. Chlorine.	Parts per Million. Free Ammonia.	Parts per Million. Albuminoid Ammonia.
1	Sayaji Sarovar, Ajwa .	14·0	7·0	1·25	0·1000	0·35[1]
2	Settling tanks, Nimeta	13·3	7·0	1·25	0·095	0·28[2]
3	Covered service reservoir, Nimeta . .	11·2	6·3	1·20	0·013	0·16[3]

[1] Sediment: Large vegetable and mineral débris, *paramecia.*

[2] Sediment: Pretty large, consisting of vegetable and mineral débris. No *infusoria.*

[3] Sediment: Scanty, consisting of vegetable débris. No *infusoria.* Good fit for drinking purposes.

TABLE II.—ANALYSIS OF FILTERED WATER FROM STAND-POSTS IN THE CITY OF BARODA, BY DR. D. R. DHUME.

Dates of Collection of Water.	Locality.	Total Solids. (a) Before incineration. Grains per Gallon.	Total Solids. (b) After incineration. Grains per Gallon.	Grains per Gallon. Chlorine.	Parts per Million. Free Ammonia.	Parts per Million. Albuminoid Ammonia.	Sediment
1892.							
31st May	Stand-post	16·1	7·7	1·5	0·040	0·20	None.
27th June	Roopura stand-post	11·2	4·9	1·5	0·106	0·24	None.
27th July	Leheripura stand-post	14·0	9·1	1·5	0·072	0·31	Scanty.
31st Aug.	Mint stand-post . .	7·7	1·4	1·6	0·020	0·24	None.
28th Sept.	Ghadiyali Pol stand-post . .	9·8	4·2	1·3	0·026	0·20	None.

TABLE III.—ANALYSIS OF WATER COLLECTED FROM THE COVERED SERVICE-RESERVOIR AT NIMETA, BY DR. D. B. DHUME.

No.	Dates.	Grains per Gallon.			Parts per Million.	
		Total Solids.		Chlorine.	Free Ammonia.	Albuminoid Ammonia.
		(a) Before incineration.	(b) After incineration.			
1	11th April, 1892 .	11·2	6·3	1·2	0·013	0·16[1]
2	4th November, 1892	11·2	4·9	1·25	None	0·14[2]

[1] Sediments scanty, consisting of vegetable débris. No *infusoria*. Good, fit for drinking purposes.

[2] Sediments very scanty, a few minute *zöospores*.

(*Paper No. 2734.*)

"The Water-Supply of Jeypore, Rajputana."

By Colonel Samuel Swinton Jacob, C.I.E., Assoc. Inst. C.E.

(*Abridged.*)

The city of Jeypore was founded in the year 1718 A.D. by Maharajah Sewaie Jey Singh. It is situated in a small valley 5 or 6 miles long, surrounded, except on the south, by hills. The soil between the hills and on the plain is drift sand, interspersed here and there with beds of kunkur, or limestone nodules, which are found generally a few feet below the surface. A small stream called the Amani Shah, Fig. 1, Plate 4, which rises in the hills north of the city and flows past it about 1½ mile to the west, was evidently at one time dammed or "bunded" and diverted to the city.

That the supply obtained from this stream, supplemented by wells in the city, proved insufficient for the needs of the inhabitants is indicated by traces that are found of attempts having been made to bring water from a river called the Brandi, about 25 miles to the west. These works, however, appear to have been unsuccessful, and surveys made lately show that there was no gathering-ground which could have been depended upon for a proper supply, nor any site suitable for a storage-reservoir. The slope of the Amani Shah is about 16 feet in a mile, and as the soil is simply drift sand, it can readily be imagined how the bed was scoured by floods. This was not all—every year, when the reservoirs formed by the earthen dams across the stream were full, as there was no proper provision for discharging the waste water, the banks were overtopped and carried away—the bed being scoured to a considerable depth as a natural consequence. The expense and labour involved by the repeated renewals of the earthen dams led at length to a masonry dam being constructed across the river.

Old Impounding-Reservoir.

This dam was founded on wells, and appears to have been built of first-rate masonry; it was about 60 feet high and 300 feet in length, with a massive stepped apron to discharge the surplus

water. Masonry steps were built on the banks of the nullah at each end of the dam on the up-stream side to afford access to the water; and wells for irrigation were made along the banks of the storage-reservoir. A masonry conduit, 3 feet by 2 feet in cross-section, provided with vertical air-shafts at intervals of 400 feet, was constructed for a length of 3 miles to the city, where open reservoirs were built in the two public squares to receive the water. After the completion of the dam, the reservoir took some seasons to fill, but on the eve of the inauguration of the supply, the dam, which had cost 4½ laks of rupees, gave way, and in a brief space of time the lake was emptied and the work was utterly ruined; affording, in the words of the late Maharajah Sewaie Ram Singh, who was an eye-witness of the catastrophe, "the grandest and most expensive spectacle he had ever seen."

In design and construction the work was sufficient for any site with a rock foundation. Its failure was due to a work of that character being built in a nullah with sandy bed and banks, and to the wings not being carried far into the banks. The water appears to have found a passage round the west end of the dam, and perhaps under the foundation also.

No further attempt was made to supply the city with water until, in 1873, the project described in this Paper was undertaken. After an exhaustive inquiry into the feasibility of other schemes, it was decided to obtain a supply from the Amani Shah nullah, the water of which was pronounced by the Government analyst at Calcutta to be "of excellent quality." An anicut or weir was thrown across the bed of the nullah at the site of the old broken dam (Fig. 2, Plate 4). This was a masonry wall 6 feet high and 3 feet thick, founded on rectangular wells of masonry 9 feet by 5 feet, sunk 6 feet deep and filled with concrete, with intervals of 6 inches between them, to prevent them from jamming against each other whilst being sunk. On the down-stream side broken material from the old dam and rubble were spread to form an apron sloping 1 in 12, reaching to within 2 feet of the top of the weir. It was further protected by a pavement of heavy schistose slabs, 10 feet to 12 feet long, connected together by a ¾-inch iron chain which is secured to the wings at each end.

A pumping-house was erected containing two pairs of 11-HP. horizontal rotative high-pressure steam-engines, with 12-inch cylinders of 24-inch stroke, each pair of engines driving two sets of 9½-inch three-throw plunger-pumps, capable of delivering 36,000 gallons per hour to the city. Steam was supplied from two Root boilers. The flue, which had a sectional area 20 square feet, was

taken up the sloping bank to a masonry chimney at the top, the total height being about 86 feet. These works were completed in 1874, and answered all expectations; but the demand for water annually increased, until during May and June, 1881, the water in the stream was only just sufficient to meet the demand.

In 1883, the supply ran so short that an intermittent system of distribution had to be introduced. It was then decided to sink a well in the bed of the stream, near the engine-house, and to erect an auxiliary pump over it, with a view to tap the springs below when the surface-water failed. The well was of masonry, 20 feet in internal diameter, carried on a wrought-iron curb 4 feet high and 3 feet 8 inches wide at the top, filled with concrete. Eight $1\frac{1}{4}$-inch bolts were carried up from the curb in the middle of the masonry, and at every 20 feet there was a horizontal band of flat-iron, 3 inches $\times$ $\frac{1}{4}$ inch, and with a band of iron outside to brace the structure together. Weep-holes 15 inches by 12 inches were left in the masonry about 8 feet apart horizontally and 2 feet vertically. These were closed with faces of dry bricks inside and outside, and the spaces between them were filled with sharp sand. The object of the weep-holes was to permit free infiltration of water through the sides of the well, and so to diminish the tendency of sand to blow up from the bottom. Owing to the approach of the hot season, sinking was stopped at 80 feet when there was 68 feet depth of water in the well, and a Davey pumping-engine was erected on the top, and was worked when required. Notwithstanding this, in 1884 it was again found necessary to restrict the supply to the city during May and June.

The necessity of removing once for all this trouble, and the fact that during the rains a large quantity of water flowed away, led to its being decided to store this surplus water for use in the hot weather. The obstacles which presented themselves in dealing with the proposal were: that the bed and banks of the river are composed of loose sand, and the difficulty of arranging outlet-works and an efficient and safe waste-weir. It must also be observed that the banks of the river are about 700 feet to 800 feet wide at the top and 61 feet deep; and any attempt to build a dam had to be completed in one season between the rains.

New Impounding-Reservoir.

A site was selected 750 feet above the pumping-station, where an embankment thrown across the river would impound 148 million cubic feet of water up to the 46-feet contour. The drainage-area

above this point is about 13 square miles. The mean rainfall is 24 inches, but considering the nature of the soil, it was not expected that more than 4 inches of rain, or about 120 million cubic feet of water, would flow off annually. If, however, from silting of the reservoir or any unexpected cause the water should ever reach the 46-foot contour, provision was made for its escape by cutting a channel 3,795 feet long at that level, leading to low ground on the west. The bed-slope of this channel was 1 foot in a mile, the bottom-width being 20 feet, and the side-slopes 1 to 1; and any overflow would pass away by it gradually, over the natural surface of the land. The experience which has been gained since the embankment was made has shown that only about one-sixtieth part of the rainfall flows off the catchment-area into the reservoir. This, no doubt, is largely owing to the absorbent nature of the soil, as one-tenth part has been found to be the average elsewhere in the district.

The embankment possesses some points of interest, in that it has no core-wall; that it is made of sand, resting upon sand and mud, the natural surface being merely dug up and coarse grass roots being removed.

Its dimensions are:—

	Feet.
Length at top	680
Height	61
Breadth at the top	30
" " base	390
Inner slope	4 to 1
Outer "	2 to 1

Work was begun on the 27th June, 1884. Light rails, 16-inch and 24-inch gauge, and double side-tipping wagons were ordered by telegraph from England, and, as soon as they arrived, the work was vigorously prosecuted. Sidings were laid down on each bank with a slight incline. Two men were placed in charge of each wagon, which, after being filled, was pushed along, and soon acquired sufficient momentum to run on to the site of the bund, carrying both the earth and the men also, who quickly jumped on to the wagon-frame when it started. As the embankment rose the rails were also raised, and the speed and economy with which the work was done were highly satisfactory. As many as 129 wagons were at one time employed, bringing about 30,000 cubic feet of sand daily from a distance of about 1,000 feet, at half the cost of manual labour. Extra men were employed to spread and ram the earth, and a few Raj elephants walked backward and forwards, morning and evening, over the work to consolidate it.

A temporary outlet, consisting of two 12-inch sluices, was built 5 feet above the river-bed at one side, and the nullah was closed on the 26th October, with the object of impounding as much water as possible for the ensuing hot season. In the meantime the permanent outlet-well and culvert were taken in hand. The difficulty of laying the foundations was increased owing to the water, which by this time had risen to a height of 13 feet. The foundations were laid on hollow wooden frames, the outside dimensions of which were 13 feet by 8 feet, on which masonry walls were built 6 feet high and 1 foot 6 inches thick. When these were thoroughly set, they were sunk, and the internal spaces were then filled with concrete (Fig. 3, Plate 4). An interval of about 1 foot was left between each pair of frames to prevent jamming during the process of sinking. These were afterwards filled with concrete, and the whole surface was covered with slabs of stone.

The outlet-well, Fig. 3, is of masonry, oval in plan, 11 feet 6 inches by 9 feet 6 inches inside, carried down about 12 feet below the ground and built to a height of 40 feet above it. Wing-walls are provided on the water-side, with cross-walls to counteract any thrust. In these cross-walls are large openings over which movable iron gratings are fixed. The water passes through these gratings to two 12-inch outlet valves, which have gun-metal faces and are raised or lowered by vertical iron rods, worked from cast-iron pillars at the top of the well. A flight of stone steps outside gives access to the top of the well, which is covered with a wooden floor protected by a handrail round it. A masonry staging is built above the sluices, carrying a travelling crab-winch from which a dredger can be worked to remove any silt that may accumulate in front of the inlet-valves; stone slabs are fixed inside at convenient distances to form a staging for a light iron ladder to be placed when required, to allow of descent and access to the valves. Inside the well are two sluice-valves similar to the two outside it. These admit the water to two 12-inch flanged pipes, which are laid through the culvert side by side. In order to prevent sand from blowing up inside, the bottom of the well was closed with a wedge-shaped mass of concrete. The outlet-culvert, a cross-section of which is given in Fig. 4, is of masonry, 7 feet wide and 7½ feet high. At the end and half-way, air-shafts are built for purposes of ventilation.

It was feared that when the reservoir filled, water might creep along the culvert; and to prevent any chance of this, slabs of sandstone, 3 inches or 4 inches thick and about 8 feet long, were built in the masonry of the culvert, projecting 6 feet all round it,

forming collars at every 50 feet along the length of the culvert, against which the earth was well rammed, so that any leakage should be prevented from creeping along the surface of the masonry.

A common source of weakness in earthen bunds occurs at the toe of the outer slope, where the earth often weeps with leakage and the slope of the bank becomes a hollow curve. To obviate this in small embankments, the natives often drive in stakes, and weave about them a wall of thorns or coarse grass to prevent earth from coming away with the leakage. In the present case, the following course was adopted:—Next to the earth a layer of sharp sand, about 10 feet wide and 5 feet deep, was placed; outside this a similar layer of small broken stone; and finally a similar mass of large rubble. This was carried along the toe of the outer slope. It was anticipated that the whole of this material might sink and disappear in the first year, and that it might be necessary to renew it more than once; but that when it did stand, it would act as a filtering medium, keeping back the earth, but allowing water to percolate out free from silt. It has answered better than was anticipated, but little settlement has occurred, and it has not had to be renewed. There is less leakage than was expected, and the water passes away quite clear. Springs occur even at the foot of the side banks of the nullah, 200 feet or 300 feet below the site of the dam, showing how porous the soil is. Another danger in large earthen bunds is that the rain-water which falls upon the surface of the embankment often drains towards some low point, and then, running down the slopes, cuts deep gutters. To prevent this the following plan was adopted: On the top of the dam a low bank of earth was made on both sides, and the top was rammed with kunkur. This formed a footway along each side, the centre portion being left unmetalled; and any water which falls upon the top of the embankment during the rains sinks quietly into it and disappears without doing any harm. The inner slope is covered with a 9-inch layer of broken stone up to high-water mark; above this and on the outer slope coarse grass is planted.

The work was completed in September, 1885. The greatest flood occurred on the 1st August, 1885, when the water rose 5 feet 10 inches in twenty-four hours. On the 1st September, the level of the water was 24 feet 4 inches, and on the 31st December 23 feet 11 inches. The water generally reaches its highest level in December or January, showing that the reservoir is fed by springs. The highest level yet attained is 31½ feet.

PUMPS AND FILTER.

With the view of utilizing some of the surplus water, the two 12-inch pipes which are laid through the culvert were united and connected with a Schiele vertical turbine 2½ feet in diameter, which was placed in the deep well. By means of gearing, three 7-inch horizontal rams, of 15 inches stroke, are worked by it at 20 strokes per minute, delivering about 6,600 gallons per hour into the service-reservoir, 110 feet above it.

The average daily consumption of water had by the year 1890 increased beyond the capabilities of the pumping-engines that were erected in 1874; and in 1892 an additional engine and pump was erected, capable of raising 2,000,000 gallons per day. This machinery, which was made by Messrs. Easton and Anderson of Erith, is a compound beam-engine, the two bucket- and plunger-pumps being driven off the beam on either side of the centre. The sizes of the cylinders and pumps are:—

	Diameter.	Stroke.
	Inches.	Inches.
High-pressure cylinder	16¼	40
Low-pressure cylinder	25	60
Bucket pumps	20½	30
Plunger „	$14\frac{5}{16}$	30

The two 9-inch pipes which were laid down for the old engines were taken up, and a single 16-inch pipe was substituted for them, the delivery-pipes from the old engines being connected with this new main. Coal is obtained from mines near Calcutta and costs Rs.32 4 0 per ton delivered at Jeypore, an import duty of Rs.10 11 0 per ton being levied on it by the Jeypore Durbar. The boiler-house consists of merely a thatch roof, open along the ridge, carried by light iron trusses supported on masonry pillars, the sides being open. This is quite sufficient for its purpose.

The filter is supplied by an open masonry conduit from the anicut, Fig. 2, Plate 4, and, as soon as the water in it is 2 feet in depth, the surface is level with the crest of the anicut which serves as an overflow. The area of the filter is 160 feet by 80 feet; its section is thus composed (Figs. 5 and 6):—

	Feet.	Inches.
Water	2	0
Fine sharp sand	2	0
Coarse sand	0	3
Broken stone ¾-inch to 1½-inch gauge	1	0
Covering-slabs and drain	0	6
	5	9

The filtered water passes into a small covered tank, from which it is drawn by the pumps. The filter was designed to allow of sufficient water passing through, at the rate of 6 inches per hour, to keep the pumps supplied. The supply to the filter can be shut off at any time, and valves communicating with the river allow it to be emptied when it is required to clear it. A simple arrangement of valves on the suction-main renders it possible to draw the supply from the filter or direct from the river as may be desired.

Service-Reservoirs.

The service reservoirs (Figs. 7, Plate 4), in duplicate, are placed on the highest ground in the neighbourhood, about 2,000 feet distant from the pumping-station. The floor of these reservoirs is 103 feet above the pumps and 36 feet above the city squares. Each reservoir is 150 feet by 100 feet at the bottom, by 15 feet deep, and contains 1,477,000 gallons. When the reservoirs were built they were open, but it was found that by exposure to the sun vegetable life was rapidly developed in the water. They were then covered by light masonry arches resting on masonry pillars. After that was done all signs of vegetable growth disappeared. The water is admitted from the 16-inch rising main into a sheet-iron vessel with one outlet, which can be turned round opposite to the reservoir into which it is desired to deliver water. One reservoir is in use for the supply to the city while the other is being filled; and by noting the depth of water daily, it is easy to calculate the consumption. The outlets are 12-inch screw-down valves with gun-metal faces, the mouths of the outlet-pipes being protected by large cages of finely perforated zinc, on iron frames which can be easily removed at any time.

Aqueduct and Distribution-System.

A 12-inch cast-iron spigot-and-socket pipe conveys the water to the city, Fig. 1, Plate 4, where it is distributed by pipes of smaller dimensions to the palace, the streets, the public gardens, and some of the public buildings. A 6-inch main supplies the Imperial Service Jeypore Transport Corps, and thence a 3-inch pipe is laid to the State Cotton Press. Another 6-inch main is laid direct to the private residence of H.H. the Maharajah. Scour-valves are placed at the lowest points on the line of pipes and at all dead-ends, and these are opened about once a week to clean out the pipes. All pipes above 3 inches in diameter are of cast-iron, those

of smaller size for house-connections being of galvanized wrought-iron. There is a stop-valve for every street. Stand-posts have been erected at the corners of all streets which intersect the main line of pipes. These are placed generally about 20 feet distant from the main, and a stop-valve is fixed on the branch so that the stand-post can be disconnected at any time. Self-closing ball stand-posts were tried at first, but were found to be unsuitable for drinking purposes for natives. The stand-post which is found to answer best here is a 4-way cast-iron post; two ¾-inch taps are provided for filling vessels, and two others for drinking purposes. The latter are fitted with diaphragms in which are small holes which allow the water to escape as fast as a man can conveniently drink. A stone step at the base allows one foot to be raised, so that a water-vessel can be rested on the knee whilst being filled.

No water-rate is levied on the city; the cost is met from funds provided by His Highness the Maharajah as a free gift to the people. Only those persons pay who have water laid on to their houses. To these the following rates are charged:—

	Rs.	a.	p.	
For the first tap	1	0	0	per month.
If used for watering cattle	2	0	0	„
For the second and every other tap	0	8	0	„
For a drinking-tap *pro bono publico*	5	0	0	„

Rs.5 is the highest charge made. For this, the payer can have as much water as he wants, except for garden purposes, for which it is not allowed to be used. The water is not allowed to be used in watering the streets, as these can be watered at less cost with water drawn from the existing wells at the road-sides. The average daily consumption has increased steadily from 263,988 gallons in 1877–78 up to 897,884 gallons in 1892 (Appendix). In order to ascertain the amount of water used per head, all sources of supply were watched simultaneously on one day, and the amount thus taken showed, on the basis of the 1891 census, an average daily consumption per head of 7·6 gallons, at a cost, excepting charges for interest and depreciation, of about 2½ annas per 1,000 gallons.

The cost of maintenance, from the returns for the year 1892, is:—

	Rs.	a.	p.
Establishment	9,266	6	0
Fuel	41,125	6	9
Sundries	1,486	12	0

The establishment consists of a European engineer, a European assistant engineer, a mistri or foreman, three native drivers, a

smith, a boiler-foreman, six firemen, six oilmen, four men and ten apprentices. These men work in shifts of eight hours, as they have sometimes to work day and night.

The expenditure on the works to date has been—

	Rs.
Weir	9,092
Pumps	133,853
Buildings	82,216
Boilers	34,341
Supply wells	65,765
Turbine	7,500
Filter-beds	19,615
Amani Shah storage-reservoir	107,454
Service-reservoirs	79,013
Pipes	415,511
Miscellaneous	43,249
Total	997,609

The works were designed and carried out by Colonel S. S. Jacob, Assoc. Inst. C.E., of the Public Works Department, whose services were lent by the Imperial Government to the Jeypore State in 1867, and who has been employed there since that time. He has received assistance in consultation from Mr. J. W. Gray, M. Inst. C.E., Birmingham; from Messrs. J. C. and W. Lord, of Birmingham, who, from time to time, have been referred to, and have helped to carry out all the work required from England; and from Mr. J. Dominy, the Engineer of the Jeypore Waterworks, whose advice has been highly valued by the Author.

The Paper is accompanied by six tracings, from which Plate 4 has been prepared.

APPENDIX.

STATEMENT SHOWING AVERAGE DAILY CONSUMPTION OF WATER, 1877–1892.

Year	Average Daily Consumption in Gallons.	Year.	Average Daily Consumption in Gallons.
1877–78	263,988	1886	605,573
1878–79	310,512	1887	635,313
1879–80	359,364	1888	660,882
1880–81	416,694	1889	722,556
1881–82	384,858	1890	830,854
1882–83	483,371	1891	881,099
1884	518,002	1892	897,884
1885	560,890		

(*Paper No. 2718.*)

"On the Design of Masonry Dams."

By Franz Kreuter, Professor of Civil Engineering in the Royal Technical Academy of Munich.

Preliminary.

The determination of the proper cross-section of a masonry dam has hitherto only been effected by a series of trials accompanied by tedious calculations. The Author proposes to show how, by dividing the problem into several distinct parts, and by introducing certain limitations and suppositions, trials may be dispensed with, and a mathematical solution may be gradually arrived at as exact as may be desired.

The calculations in question, moreover, are in no way more complicated or difficult than those in actual use for determining the stresses in other large engineering structures, such as metal bridges.

I. Origin and Present State of the Theory of Masonry Dams.

The idea of giving to the cross-sections of masonry dams a shape of uniform stability and strength is of French origin, and was first published by Mr. de Sazilly in the "Annales des ponts et chaussées," 1852.

The first solution of the problem, at once scientific and practical, was given by the French engineers Messrs. Graeff and Delocre, who built the Furens dam, the largest masonry dam in existence until recent times.[1]

Delocre, starting from de Sazilly's principles, was led to a standard shape of cross-section which has ever since been used as

[1] Graeff, "Rapport sur la forme et le mode de construction du barrage du Gouffre d'Enfer," &c., "Annales des ponts et chaussées," 1866, p. 184.

Delocre, "Sur la Forme du profil à adopter pour les grands Barrages en Maçonnerie des Reservoirs," *ibid*, p. 212.

a model, and has been usefully developed by another eminent French engineer, Mr. Krantz, who, in 1870, published a selection of types of reservoir-banks, designed according to the principles established by Sazilly and Delocre.

An interesting and valuable report on the design and construction of masonry dams, by Professor Rankine, was published shortly before his death,[1] and was probably one of the last Papers issued by that unrivalled investigator. Rankine added two very essential conditions to those established by de Sazilly and accepted by Delocre, so that the basis for any sound theory of these structures may now be stated as follows :—

1. That, at any horizontal section, the intensity of pressure at the faces of the wall shall never exceed a certain value fixed upon as the safe crushing load of the material of the dam.

2. That at no horizontal layer of the masonry shall there be any danger of sliding.

3. That at those parts of the profile where the wall has a batter, the intensity of pressure at the faces shall be diminished below the limits answering to vertical faces.

4. (A principle not referred to by the French authors.) That there ought to be no practically appreciable tension at any point of the masonry, whether at the outer face, when the reservoir is empty, or at the inner face, when it is filled. The lines of resistance therefore should not deviate from the middle of the thickness of the wall to an extent exceeding one-sixth of the thickness.

Lastly, mention should be made of an excellent Paper of the late Professor Harlacher, of Prague,[2] Austria, containing full information for calculating cross-sections of reservoir-walls by means of graphic statics.

Now, until recently, whenever it has been attempted to design a reservoir-wall of an economic cross-section, the French types have served as models, modifications being made to meet the special cases. Papers on the subject published lately have scarcely contributed any new points of view or appreciably forwarded the theory of masonry dams, so that, in comparison with the progress made in the theory of structures in general during the last forty years, the results arrived at by the process now in use are sensibly inferior in exactness to those obtained in other departments of modern statics. By the present communication the Author hopes to make a modest step in advance.

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[1] *The Engineer*, 1872, p. 1.

[2] Published in "Technische Blätter," 1875, pp. 89 and 169.

II. Principles of the New Theory.

A. *The following assumptions are made:*—1. The water-level is supposed to reach to the top of wall.

2. The vertical component of the water-pressure upon the battered part of inner face of the wall is provisionally neglected.

3. The shearing-stresses acting parallel to the layers of the wall are not allowed for.

For the rest, the principles as established by de Sazilly and Rankine, and mentioned in the preceding paragraph, will be adhered to.

The first and second of the above assumptions at once serve to simplify the calculations and to favour safety. The third is of no importance when the wall is properly executed according to the rules first established by Graëff and accepted by Krantz and Rankine.

The first supposition generally corresponds with the most unfavourable case that can occur, when the pressure of water is greatest. A quantity of mud occasionally collected at the bottom of the reservoir, indeed is apt to exert a pressure exceeding that of pure water; but this quantity, of course, never should attain a considerable depth, and moreover it must not be forgotten that the working water-level is always kept considerably below the top of the wall.

The margin of safety given by adopting the first assumption may be calculated as follows:

Let x be the vertical depth of any point in the face of the wall below the top of the wall, and x' the depth of the same point below the actual water-level; let w' be the weight of unit of volume of water. Then the actual moment of the water-pressure on the wall above any horizontal section is $\frac{w' x'^3}{6}$, whereas the moment deduced from the assumption of the reservoir being full is $\frac{w' x^3}{6}$. This assumption gives to the wall, therefore, at the cross-section in question, a factor of safety of $\left(\frac{x}{x'}\right)^3$.

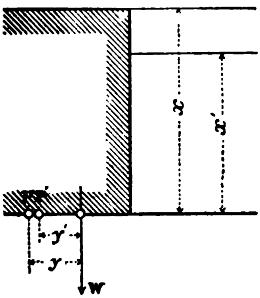
As x and x' increase, this factor diminishes and tends to the limit unity. On the other hand, its value increases in approaching the upper part of the wall, where such a factor is more valuable, owing to the smaller mass to resist impacts from waves or ice.

When the cross-section of the wall has been determined on this

assumption, and when its line of resistance is known, it is an easy matter to find the line of resistance of the same wall for any given lower water-level.

Thus, let P and P′ be the centres of pressure in the two cases respectively for any horizontal plane, y and y' the distances of P and P′ from the vertical through the centre of gravity of the part of the wall above that plane (*Fig. 1*), W the weight of the same part of the wall.

Fig. 1.



Then the moment about P of the weight W is W y, and this is equal to the moment of the water-pressure, which, as stated above, is $\frac{w' x^3}{6}$.

Similarly

$$W y' = \frac{w' x'^3}{6}.$$

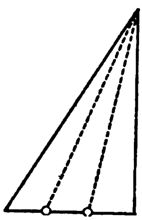
Therefore

$$\frac{y}{y'} = \left(\frac{x}{x'}\right)^3,$$

and y being known for every cross-section, y' can be calculated. The second limitation, consisting in at first neglecting the vertical component of water-pressure acting on those parts of the inner face of wall which are not vertical, besides simplifying the calculations, offers the advantage that in the lower parts of the wall, where the influence of the ratio $\frac{x}{x'}$ considerably diminishes, the line of pressure, corresponding to the reservoir filled, will be displaced towards the middle, and consequently the intensity of the vertical pressure at the outer slope of the wall will diminish as the outer slope of the walls flattens towards the bottom.

It will be shown, however, that the vertical component of the water-pressure may be allowed for in cases where it may be found to be desirable.

Fig. 2.



B. *Primary Shape of Cross-section.*—The simplest among appropriate cross-sections, namely, those of uniform stability throughout, is a right-angled triangle, as in *Fig. 2*, with vertical inner face.

Then the thickness of the wall at any horizontal cross-section is given by the equation[1]

$$\frac{t}{x} = \sqrt{\frac{w'}{w}}$$

[1] Rankine "Applied Mechanics," 4th ed., p. 247.

where t is the thickness of the wall at the cross-section whose depth is x, and w is the weight of unit volume of masonry.

The lines of resistance for the reservoir full and empty are both straight lines, trisecting each horizontal cross-section of the wall, and the pressures at the faces of the wall in the two cases increase proportionally to the depth. This form of cross-section, however, cannot be realized in practice owing to the necessity of giving a certain width to the top of the wall, and often a sufficient width to form a road. The uppermost part of the wall is frequently a rectangular block of masonry, and where it is not so, it may be represented for the purposes of design by such a block, equivalent to the actual structure; *i.e.*, the equivalent block will be of the same weight as the actual structure superposed on the dam, and will have its centre of gravity in the same vertical line.

If such a superstructure be placed upon the triangular dam above considered, the lines of resistance will be displaced throughout the whole body of the dam. If, then, it is desired that the condition of uniform stability should prevail, that is to say, that the conditions laid down above (p. 64) should be fulfilled by every horizontal cross-section to the same extent, it becomes necessary to depart from the triangular cross-section, and among the elements determining the cross-section to be sought will be the mass and form of the superstructure. Delocre recognized this necessity and divided a reservoir-wall into three portions, to which different methods were applied; namely (1) the rectangular superstructure, (2) the body of the wall with vertical inner face and curved outer face carried down to a level where the pressure at the inner face with the reservoir empty becomes equal to the maximum pressure permissible, and (3) the lowest portion or base of the wall with both faces curved.

It is well known that if the weight of the masonry alone were to be considered, the faces of this third or lowest portion of the cross-section would be logarithmic curves, and it will be shown hereafter that for the case under consideration the curves will be of the same class.

This division of the subject as given by Delocre is incomplete, because if the same conditions of stability are made to apply throughout, there will be a discontinuity at the plane of junction of the portions Nos. (1) and (2), as will be seen at once by considering what those conditions are.

In the portion No. (1), which is a rectangular block either forming the actual superstructure or equivalent to it, the only condition to be fulfilled is that the line of pressure with full

reservoir must be at a distance from the outer face of the wall of at least one-third of the thickness. With the reservoir empty the line of pressure is vertical, bisecting the thickness. In the portion No. (2) the lines of pressure for the reservoir full and empty must trisect each horizontal cross-section. It is therefore necessary to introduce between the portions Nos. (1) and (2) another portion, to which the condition of No. (1) will apply at its upper limit and those of No. (2) at its lower limit.

The following investigation is therefore divided into four parts treating respectively of the four portions of the wall to which different conditions have to be applied. These portions will be hereafter referred to as Nos. (1), (2), (3) and (4), of which Nos. (3) and (4), correspond to Delocre's divisions Nos. (2) and (3) above mentioned, with this difference, that the inner face of the wall is not assumed to be vertical in portions Nos. (2) and (3), although it is nearly so.

III. Calculation of Cross-section.

The process of determining the cross-section may be divided into three stages, namely: (A) to determine the thickness of the wall throughout from top to bottom, but without regard to the form of the inner or outer faces: (B) knowing the thickness of the wall at every horizontal section, to determine the right profile for the faces: (C) to make allowance in the figure of the cross-section, if need be, for the vertical component of the water-pressure on the inner face.

A. To determine the thickness of the wall throughout.—*Portion No. (1).* Let the width of the top of the wall be given and be t_0. Then the height, a, of a rectangular block, such that at its base the line of pressure with the reservoir full falls at one-third of the thickness of the wall from the outside, is given by the equation[1]—

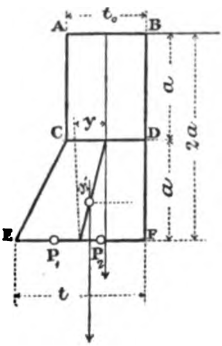
$$\frac{a}{t_0} = \sqrt{\frac{w}{w'}} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

If the wall were carried down with vertical faces to a greater depth than this, the centre of pressure would be within the outer third of the thickness, which (on the assumption of uniformly varying pressure) means that there would be a tension in the masonry at the inner face.

[1] Rankine, "Applied Mechanics," 4th ed., p. 247.

Portion No. (2). This portion has only to fulfil the conditions of portion No. (1) at its upper limit and of portion No. (3) at its lower limit. In other respects the thickness is arbitrary so long as the lines of resistance do not pass outside the middle third. The simplest form to take satisfying these conditions is the trapezoidal. Let A B D C (*Fig. 3*) be the superstructure and C D F E a trapezoidal block forming portion No. (2). The slope of C E and D F is at present quite arbitrary. The height of the trapezoid is also arbitrary, since by choosing the slopes suitably the conditions at the upper and lower limits can be satisfied with any height. The height must not, however, be taken too small, as this would require a considerable batter to be given to the inner face, which is by hypothesis nearly vertical. A convenient height to take, and one having the further advantage of simplifying the calculations, is that equal to the height of the superstructure or a. Let t be the thickness at the base E F. The weight of masonry, then, resting on E F for unit of length of the wall is

Fig. 3.



$$w\{a\,t_o + \frac{a}{2}(t_o + t)\} \text{ or } \frac{w\,a}{2}(3\,t_o + t).$$

Let P_1 and P_2 be the points of trisection of E F.

The centre of gravity of this weight is by hypothesis vertically above P_2, and therefore its moment about P_1 is $\frac{w\,a}{2}(3\,t_o + t) \times \frac{t}{3}$, while the moment of the water-pressure is $\frac{w'(2a)^3}{6}$.

Since P_1 is the centre of pressure for the reservoir full, these moments are equal; therefore

$$w\,t\,(3\,t_o + t) = 8\,w'\,a^2.$$

But since, by equation (1), $w'\,a^2 = w\,t_o^2$, it follows that

$$t^2 + 3\,t_o\,t - 8\,t_o^2 = 0,$$

the solution of which equation gives

$$t = 1 \cdot 7\,t_o \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Portion No. (3.) Let t be the thickness of the wall at a depth x below the top, A the total area of the cross-section to that depth.

Taking moments exactly as was done for portion No. (2) we have for any horizontal cross-section at depth x—

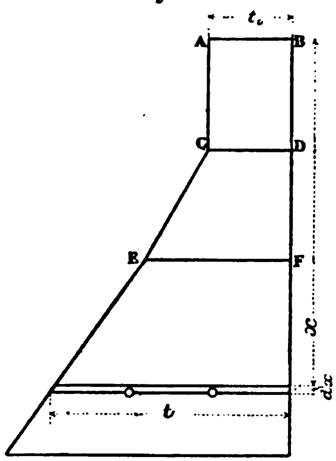
$$w\,A \times \frac{t}{3} = \frac{w'\,x^3}{6}.$$

Let $$\frac{w}{w'} = \rho.$$

Then $$2\,A\,t = \frac{x^3}{\rho} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

If x increase by a small increment $\Delta\,x$, $\Delta\,A = t\,\Delta\,x$, and in the limit when $\Delta\,x$ is infinitely small, $t = \frac{d\,A}{d\,x}$, whence the preceding equation becomes

Fig. 4.



$$2\,A\,\frac{d\,A}{d\,x} = \frac{x^3}{\rho}.$$

Integrating, $A^2 = \frac{x^4}{4\,\rho} + C$, where C is the constant of integration.

Let A_o be the area of portions Nos. (1) and (2) which the preceding work enables us to calculate.

$$A = A_o \text{ when } x = 2\,a.$$

Therefore $$A_o^2 = \frac{16\,a^4}{4\,\rho} + C,$$

and $$A^2 = A_o^2 + \frac{x^4 - 16\,a^4}{4\,\rho}.$$

Substituting this value of A in equation (3) we obtain:

$$t = \frac{x^3}{\sqrt{\rho\,(x^4 - 16\,a^4 + 4\,\rho\,A_o^2)}}. \quad . \quad . \quad (4)$$

As x increases, t tends to the value $\frac{x}{\sqrt{\rho}}$, which is that of the primary triangular form. The curves for the faces of the wall given by (4) have the sides of the triangle as asymptotes.

The limit of depth to which equation (4) can be used is reached when the pressure at the face of the wall becomes equal to the maximum permissible pressure. Let p be this pressure. Under

the conditions assumed to prevail, the maximum pressure is twice the mean pressure,

or $$p = \frac{2\,A\,w}{t} \text{ at the limiting depth,}$$

or, by (3) . . . $$p\,t^2 = \frac{x^3\,w}{\rho} = w'\,x^3.$$

This equation may be solved by trial, commencing with the value $x = \frac{p}{w}$ which applies to the primary triangle.

If the slope of the outer face is allowed for in the limiting pressure (as is done by Rankine), $p \cos \delta$ must be substituted for p, where approximately

$$\tan \delta = \frac{x}{t} = \sqrt{\rho}.$$

Portion No. (4). The fresh condition introduced here is that the pressure at the face—*i.e.*, either at the inner face when the reservoir is empty or at the outer face when it is full—must not exceed the value p. As before, let t be the thickness of the wall at any depth, x, and A the entire area of cross-section above that depth; A_o the area of cross-section determined according to the foregoing methods for portions Nos. (1), (2) and (3). The mean intensity of vertical pressure is $\frac{w\,A}{t}$, and, on the theory of uniformly varying pressure, the pressures at the faces are $\frac{w\,A}{t} + \frac{k\,t}{2}$ and $\frac{w\,A}{t} - \frac{k\,t}{2}$, where k is a constant differing for each manner of loading. An investigation which it is needless to reproduce here, shows that the best practical result is arrived at by making the minimum pressure zero, in which case the maximum pressure is double the mean, or $p = \frac{2\,w\,A}{t}$.

As before $$t = \frac{d\,A}{d\,x}.$$

Whence $$\frac{1}{A}\,\frac{d\,A}{d\,x} = \frac{2\,w}{p}.$$

Integrating, $\log \frac{A}{C} = \frac{2\,w\,x}{p}$, where C is a constant of integration.

Therefore $A = C\epsilon^{\frac{2\,w\,x}{p}}$.

If x_o be the depth at which portion No. (3) ends and No. (4) begins, $A_o = C\,\epsilon^{\frac{2\,w\,x_o}{p}}$

Therefore $$A = A_0 \epsilon^{\frac{2w}{p}(x - x_0)},$$

and $$t = \frac{2w}{p} A = \frac{2w}{p} A_0 \epsilon^{\frac{2w}{p}(x - x_0)} \quad . \quad . \quad (5)$$

If the thickness of the wall be determined according to equation (5), one line of resistance (say that for the reservoir empty) will be everywhere at a distance from the face of one-third the thickness, but in that case the same cannot be true of the other line of resistance, because the equation of moments has to be considered. Let r be the distance between the two centres of pressure.

$$r \times w A = \frac{w' x^3}{6},$$

or $$r = \frac{x^3}{6 \rho A}.$$

This equation is of a similar form to (3), which was found for portion No. (3). As, by equation (5) above, the thickness increases very rapidly with the depth, A increases much more rapidly than x^3, and r, starting from the value $\frac{t}{3}$, constantly decreases. When r becomes less than $\frac{t}{6}$, the two centres of pressure are in the same half of the thickness.

Since the cross-section increases so rapidly with the depth after portion No. (4) is reached, it is advisable to extend the limit of depth to which portion No. (3) is carried as far down as possible, and this can only be done either by using lighter materials or by admitting higher pressures. There is not much scope for choice in the weight of the materials, but it is certainly possible to allow a greater stress than 85 lbs. per square inch, which has hitherto been taken as a limit, through the influence of French practice, although recent experiments in Germany have demonstrated the great strength of masonry laid in cement. A factor of safety of 20 to 30 seems an unnecessary precaution against crushing when the wall has a factor many times less against overturning.

B. *To determine the Configuration of the Wall, the thickness being already known throughout.*—The preceding investigation enables us to draw a provisional cross-section of the wall, having everywhere the thickness necessary to comply with the conditions of stability. The configuration is yet to be determined by certain conditions of equilibrium, which have not yet been considered. This may be

done either by analytical or by graphic methods, the former of which alone will be here dealt with.

Portion No. (1). This is already completely determined, both faces being, by hypothesis, vertical.

Portion No. (2). The position of this trapezoid, with respect to the rectangle above it, is fixed by the condition that the centre of gravity of the two together must be vertically above the point of trisection of the base nearer to the inner face. Let (*Fig. 3*) y be the distance between the middle point of E F, and the vertical through the middle point of A B, y' the distance of the vertical through the centre of gravity of the area C D E F from the same point. The area of A B C D is $t_o\, a$, and that of C D E F is (equation (2)) $a \times \frac{t_o + 1 \cdot 7\, t_o}{2}$, or $1 \cdot 35\, t_o\, a$. The area of both together is $2 \cdot 35\, t_o\, a$.

The above condition gives therefore :—

$$t_o\, a \times y + 1 \cdot 35\, t_o\, a \times y' = 2 \cdot 35\, t_o\, a \times \frac{1 \cdot 7\, t_o}{6};$$

that is,

$$y + 1 \cdot 35\, y' = \frac{2 \cdot 35 \times 1 \cdot 7}{6}\, t_o,$$

and y' is given by the formula for the centre of gravity of a trapezoid, viz. :—

$$\frac{y'}{y} = \frac{2\,\mathrm{CD} + \mathrm{EF}}{3\,(\mathrm{CD} + \mathrm{EF})} = \frac{3 \cdot 7}{8 \cdot 1} = 0 \cdot 457\,;$$

therefore

$$y\,(1 + 1 \cdot 35 \times 0 \cdot 457) = \frac{2 \cdot 35 \times 1 \cdot 7}{6}\, t_o$$

$$y = 0 \cdot 4 t_o.$$

The vertical B D overhangs the point F by the amount—

$$y + \frac{t_o}{2} - \frac{\mathrm{EF}}{2} = (0 \cdot 4 + 0 \cdot 5 - 0 \cdot 85)\, t_o = 0 \cdot 05 t_o,$$

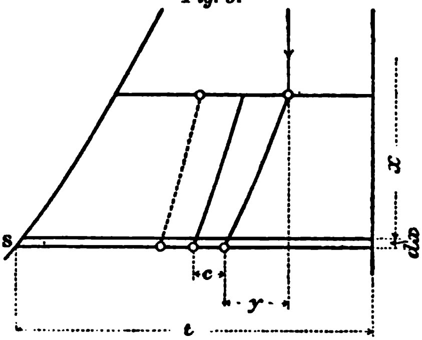
or $\frac{1}{20}$ part of the width of the wall at the top.

This trifling amount of overhang is unobjectionable from the point of view of construction, but, in order to remove any doubt, the effect has been calculated of the upward water-pressure on the overhanging face D F. It is found that the tensional stress on the masonry from this cause amounts to $90 \cdot t_o \sqrt{\rho}$ in kilograms per square metre. The value of ρ will lie between 2 and 3, and if the width of the wall on top, t_o, be even so unusually large as 5 metres, the factor of safety for good rubble masonry laid in hydraulic

mortar will be between 250 and 300, to resist this tension. It is also to be remembered that the water-level in practice does not reach the top of the wall, and that any parapet along the inner side of the wall tends to counteract the upward pressure.

Portion No. (3).—The condition determining the outline of the faces of this portion is similar to that applying to No. (2), viz., that the material must be so distributed as to conform to the rule that the line of resistance with the reservoir empty will everywhere pass at a distance from the inner face of one-third of the thickness. This line of resistance is the locus of the projections on successive horizontal cross-sections of the wall, of the centres of gravity of the whole mass above those cross-sections. Consider a lamina of thickness dx, bounded by horizontal planes.

Fig. 5.



If y be the distance from any vertical axis of the centre of gravity of the mass of the wall above the lamina, x and y are the Cartesian co-ordinates of the point where the line of resistance, reservoir empty, cuts the plane x. The thickness t being already known as a function of x, statical conditions determine dy in terms of dx. Thus, if A denote, as before, the area of the whole wall above the level x,

$$A\,dy = c \times t\,dx$$

c being the distance of the vertical through the centre of gravity of the area A from the middle point of the thickness t, and equal to $\frac{t}{6}$.

Therefore the equation to the line of resistance in question is—

$$y = \int \frac{c\,t}{A}\,dx.$$

This integration cannot be made directly, but the line of

resistance can be plotted by calculating the elements of the quantity to be integrated step by step.

The starting point of the curve at the base of portion No. (2) is known. For every element of depth $d x$, the numerator and denominator of the fraction under the sign of integration admit of calculation from the work previously done, and the course of the curve can in that way be accurately laid down.

Portion No. (4).—The condition which determines the outline, and also the method to be pursued are identical with those already stated for portion No. (3).

C. *To Make Allowance for the Vertical Component of the Water-Pressure.*—It is only in portion No. (4) that the water-pressure on [the inner face has a vertical component of any appreciable magnitude. Where, however, the wall is of such height that portion No. (4) enters into it, a valuable economy results from taking this vertical component into account. This may be approximately performed by a combined graphic and analytical process, which moreover appears to be very simple, since the two limiting cases between which the solution required is to be found may be represented with fully sufficient accuracy according to the preceding investigation.

These limiting cases are (*Fig. 6.*):

(1) Intensity of pressure at one edge, p, continuously increasing downwards, and nil at the opposite edge; that is to say, the portion No. (4) being left out and the whole cross-section, from the base of portion No. (2), E F, downwards, being calculated by equation (4), as portion No. (3). Here it is certainly allowable to neglect the vertical component of the water-pressure. This case is represented by V L E C A B D F M W in *Fig. 6.*

(2) The calculation of the cross-section, with portion No. (4) according to equation (5), without allowing for the vertical component of the water-pressure, and represented by Q L M R in *Fig. 6*, evidently represents the second, or opposite limiting case.

The intensities of the pressures at the edges Q R of the base may be obtained by calculation.

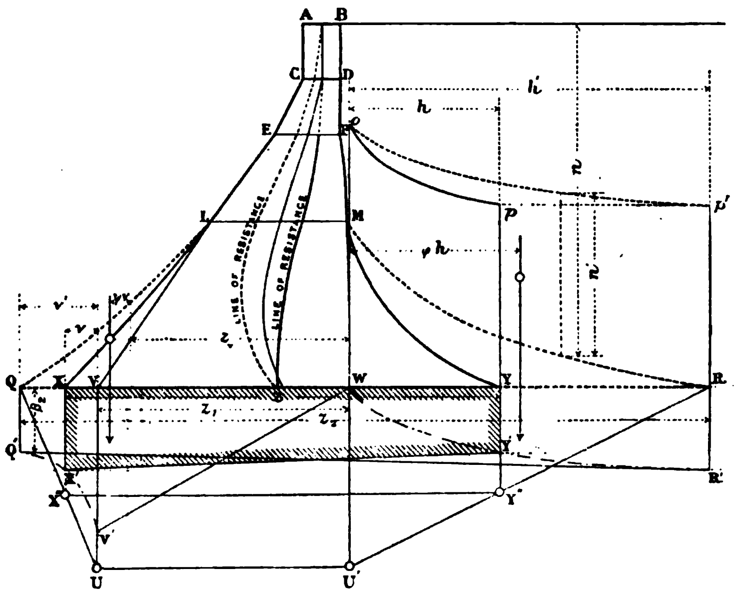
Let p_o denote the utmost pressure consistent with safety to be admitted in portion No. (4), and p_1 and p_2 pressures obtained in the limiting cases (1) and (2) respectively, then it results:—

Case.	Intensity of Pressure at the edge of Base. Reservoir filled. Outer Face.	Inner Face.
(1) Limit	$p_1 > p_o$ . . .	$p_1' = 0$;
(2) Opposite limit . . .	$p_2 < p_o$ . . .	$p_2' > 0$;
(3) Problem to be solved .	p	$p_3' < p_2' > 0$.

Now let the two profiles (1) and (2) be laid upon each other so that the upper parts, beginning from the limit L M, will coincide, as has been done in *Fig. 6*. The profiles, as well as the bases z_1 and z_2, will then have a definite relative position. In the *Fig.* $V W = z_1$ is the base of the first, $Q R = z_2$ that of the second profile.

If at the edges Q R, V W, the respective intensities of pressure be produced as perpendiculars, namely, $\overline{Q\,Q'} = p_2$, $\overline{V\,V'} = p_1$, $\overline{R\,R'} = p'_2$; by joining their proper ends, the corresponding "surfaces of load" may be obtained, the areas of which evidently must

Fig. 6.



be proportionate to the respective total loads of masonry and water.

Thus for the first case, and base z_1, there is as surface of load the triangle V V' W; for the second case, and base z_2, the trapezoid Q Q' R' R. If now the intensity of pressure at the outer edge of the base shall become p_o, the edge Q must evidently move towards V or the point Q' towards V'. At the same time, it is obvious that R as well as R' will move towards W.

The curves then described by the points Q' and R' are indicated in the *Fig.*, but will not be needed in respect of the solution of the problem.

Let the whole load of water resting on the curved inner face M R of portion No. (4) be reduced to masonry, by dividing its depths n by the specific gravity ρ of masonry, and let the lengths thus obtained

$$n' = \frac{n}{\rho} \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

be plotted vertically upwards from the curve M R and their ends be joined by a continuous curve $o\,p'$.

The spandril B D F M W o may be neglected without involving a sensible error. Then the polygon B $o\,p'$ R will represent the inner face of a masonry block, which, when acted upon by the purely horizontal pressure of the water, as already taken into account, would exert at its base z_2 a stress exactly equal to that actually arising if the profile of case (2) were executed and the reservoir were filled.

If now the process be continued from the base Q R = z_2 to a base X Y = z, lying between the limits z_2 and z_1, the point p will move horizontally from p' to p, and so will accordingly any other point of the curve $o\,p'$.

The trapezoid $o\,p$ Y W may therefore be assumed as being a parallel projection of $o\,p'$ R W, and, likewise, the triangle L X V as a parallel projection of L Q V.

Let the inner surfaces of load $o\,p'$ R W and $o\,p$ Y W respectively be denoted by H′ and H; the outer, L Q V and L X V, by V′ and V; then, using the symbols marked in the figure, there will be obtained

$$\frac{V}{V'} = \frac{v}{v'} \quad \frac{H}{H'} = \frac{h}{h'} \quad . \quad . \quad . \quad . \quad (7)$$

If, further, r signifies the mean height of the triangle L Q V, and s that of the trapezoid $o\,p'$ R W,

$$v'\,r = V', \quad h'\,s = H' \quad . \quad . \quad . \quad . \quad . \quad (8)$$

whence

$$r = \frac{V'}{v'}, \quad s = \frac{H'}{h'} \quad . \quad . \quad . \quad . \quad . \quad (8a)$$

By equations (8),

$$v\,r = V, \quad h\,s = H \quad . \quad . \quad . \quad . \quad . \quad (9)$$

Now there must still be established another relation between V and H, and it is allowable to consider the most simple case answering the purpose, that is—

$$\frac{d\,v}{d\,h} = \text{constant} \quad . \quad . \quad . \quad . \quad . \quad (10)$$

It is evident that this is sufficient to fulfil the condition, that the intensity of pressure at the outer edge shall pass through the value $x x' = p_o$, when z continuously increases from z_1 towards z_2.

Therefore there are the further following conditions—

$$\left.\begin{aligned} \frac{d\,v}{d\,h} = \frac{v}{h} = \frac{v'}{h'} = \sigma, \text{ a constant} \\ \frac{V}{H} = \frac{V'}{H'} = \frac{r\,v}{s\,h} = \frac{r}{s}\sigma, \text{ another constant} \end{aligned}\right\} \quad . \quad . \quad . \quad (11)$$

h and v can now be expressed in terms of z thus—

$$v + h + z_1 = z \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

or

$$\sigma h + h = z - z_1;$$

$$h = \frac{z - z_1}{\sigma + 1} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

and accordingly

$$v = \frac{\sigma}{\sigma + 1}(z - z_1). \quad . \quad . \quad . \quad . \quad . \quad (14)$$

Let the centres of gravity of the triangle L V Q and of the trapezoid op' R W be determined by any method; the centres of gravity of L V X and op Y W will be parallel-projections of the former.

Let, as an approximation, the lines L V and M W be taken as straight, and moreover M W as vertical.

The distance between the centre of gravity of the area H′ and the vertical M W is proportionate to the breadth h' or equal to

$$\phi\, h',$$

ϕ denoting a constant ratio, also applying for any parallel-projection of op' P W, so that the corresponding distance for the area H will be

$$\phi\, h.$$

Thus will be found the distances of the centres of gravity of the triangle L V Q and its parallel-projections, respectively

$$\psi\, v' \text{ and } \psi\, v.$$

These latter distances, however, are to be measured horizontally. The breadth of profile I, as measured at the height of the centres of gravity of the planes V (which centres obviously must be all at the same height above the base) will be denoted by z_c. Let S be the total area of cross-section of profile I; $\mathfrak{V}$, $\mathfrak{S}$, $\mathfrak{H}$, the statical

moments of the parts V, S, H of the profile to be deduced, about the middle of the base z; let the weight of S be assumed to be applied at the outer third of the base z_1: by so doing, the moment of the horizontal component of water-pressure will be allowed for; since, according to the premisses, the action of that moment consists in shifting the line of action of that weight from the inner to the outer third of the base z_1.

$\mathfrak{V}$ and $\mathfrak{S}$ obviously are positive, whilst $\mathfrak{H}$ is negative.

Now

$$\mathfrak{V} = \mathrm{V}\left(\psi v + z_v + h - \frac{z}{2}\right) w,$$

$$= \frac{r\,\sigma}{\sigma + 1}(z - z_1)\left\{\frac{z - z_1}{\sigma + 1}(\psi\,\sigma + 1) + z_v - \frac{z}{2}\right\} w. \quad (15)$$

$$\mathfrak{S} = \mathrm{S}\left(h + \frac{2}{3} z_1 - \frac{z}{2}\right) w,$$

$$= \mathrm{S}\left(\frac{z - z_1}{\sigma + 1} + \frac{2}{3} z_1 - \frac{z}{2}\right) w \quad . \quad . \quad . \quad (16)$$

$$\mathfrak{H} = \mathrm{H}\left(\frac{z}{2} - h + \phi\,h\right) w,$$

$$= \frac{s\,(z - z_1)}{\sigma + 1}\left\{\frac{z}{2} - \frac{z - z_1}{\sigma + 1}(1 - \phi)\right\} w \quad . \quad . \quad (17)$$

For determining the intensity of pressure p, at the outer edge of the base, there is the fundamental equation.

$$p = \frac{\mathrm{N}}{z} + \frac{6\,\mathfrak{P}}{z^2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

Here N, as before, denotes the total vertical pressure as distributed over the base z, and $\mathfrak{P}$ signifies the resulting moment of the whole water-pressure and weight of masonry about the middle of z. Hence

$$\left.\begin{aligned} \mathrm{N} &= w\,(\mathrm{V} + \mathrm{S} + \mathrm{H}) \\ \mathfrak{P} &= \mathfrak{V} + \mathfrak{S} - \mathfrak{H}. \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad (19)$$

It is obvious that the most favourable value of z would be obtained, if it were possible so to arrange the cross-section of bottom as to distribute the weight N uniformly over the base, that is to say, if it were possible by solving the equations of conditions

$$\frac{\mathrm{N}}{z} = p \text{ and } \mathfrak{P} = 0,$$

to obtain one distinct value of z.

This, however, as may easily be stated, is not possible in every case, when p is fixed; or in other words, these conditions will not lead to real results for any arbitrary value of p, and this is just what is required.

The process must therefore be continued as follows: finding

$$N = w\left(\frac{z - z_1}{\sigma + 1}(r\,\sigma + s) + S\right) \quad . \quad . \quad . \quad (20)$$

$$\left.\begin{aligned}\mathfrak{P} = w\Bigg\{\frac{z - z_1}{\sigma + 1}\Bigg[\frac{z - z_1}{\sigma + 1}\Big(r\,\sigma\,(\psi\,\sigma + 1) + s\,(1 - \phi)\Big)\\ - \frac{z}{2}\Big(r\,\sigma + s\Big) + r\,\sigma\,z_v + S\Bigg] + S\left(\frac{2}{3}z_1 - \frac{z}{2}\right)\Bigg\}\end{aligned}\right\} \quad (21)$$

and putting these values into equation (18) there is obtained

$$\frac{z^2\,p}{w} = z\left[\frac{z - z_1}{\sigma + 1}\Big(r\,\sigma + s\Big) + S\right] + 6\Bigg\{\frac{z - z_1}{\sigma + 1}\Bigg[\frac{z - z_1}{\sigma + 1}\Big(r\,\sigma\,(\psi\,\sigma + 1)$$

$$+ s\,(1 - \phi)\Big) - \frac{z}{2}\Big(r\,\sigma + s\Big) + r\,\sigma\,z_v + S\Bigg] + S\left(\frac{2}{3}z_1 - \frac{z}{2}\right)\Bigg\}.$$

The following quantities are next computed.

$$\frac{p}{w} = a, \frac{r\,\sigma + s}{\sigma + 1} = b, \frac{r\,\sigma\,(\psi\,\sigma + 1) + s\,(1 - \phi)}{(\sigma + 1)^2} = c, \frac{r\,\sigma\,z_v + S}{\sigma + 1} = B;$$

then, after the required transformations, there results—

$$\left.\begin{aligned}z^2\,(a + 2\,b - 6\,c) - 2\,z\Big[z_1\,(b - 6\,c) - S + 3\,B\Big]\\ = 2\,z_1\Big[2\,S + 3\,(c\,z_1 - B)\Big]\end{aligned}\right\} \quad (22)$$

Further, computing

$$a + 2\,b - 6\,c = e;\; z_1\,(b - 6\,c) - S + 3\,B = E,$$

$$2\,z_1\Big[2\,S + 3\,(c\,z_1 - B)\Big] = \mathfrak{E};$$

there is finally obtained from (22)

$$z = \frac{E}{e} + \sqrt{\frac{\mathfrak{E}}{e} + \left(\frac{E}{e}\right)^2} \quad . \quad . \quad . \quad . \quad (23)$$

By introducing into this equation p_o instead of p, the length of base required, z'_o is obtained.

It only remains to plot the latter in the right position. For this purpose equations (13) and (14) are made use of—

$$h_o = \frac{z_o - z_1}{\sigma + 1},\ v_o = \frac{\sigma}{\sigma + 1}(z_o - z_1)\ . \quad . \quad . \quad (24)$$

Now, the curve M Y is obtained by reducing in the ratio

$$\overline{\mathrm{W\,Y}} : \overline{\mathrm{W\,R}},$$

the several horizontal ordinates of the curve M R, measured from the line M W; and the curve L X is obtained by reducing in the ratio

$$\overline{\mathrm{V\,X}} : \overline{\mathrm{V\,Q}}$$

the several horizontal ordinates of the curve LQ, measured from the line L V.

It is almost unnecessary to mention, that all the calculations required are much facilitated by the use of a slide rule, or some mechanical contrivance of the kind. In Fig. 6, X L C A B M Y represents, on a scale of 1 in 1000, the shape of cross-section which would have been obtained by the foregoing theory for the well-known masonry dam of Furens; that is to say, by employing as fundamental data for the calculations

t_o = 5 metres; w = 2,000 kilograms per cubic metre,
$\rho = 2 \cdot 0$; p = 6 kilograms per square centimetre.

The greatest deviation from the above standard value of p (about three per cent.) is arrived at, at a depth of 35 metres from the top, the intensity of pressure at the outer face (reservoir filled) being there 6·2 instead of 6·0 kilograms per square centimetre; whilst at the base it returns to 6·0 exactly. This evidently arises from the neglect of the spandril B F W *o*.

At the inner face the intensity of pressure diminishes as the slope flattens towards the bottom, becoming 4·8 kilograms per square centimetre at the edge of base, Y, when the reservoir is filled, and 3·5 kilograms when it is empty.

The communication is accompanied by a series of diagrams from which the *Figs.* in the text have been prepared.

[DISCUSSION.

Discussion.

Sir Benjamin Baker.

Sir BENJAMIN BAKER, Vice-President, said it was his duty as Chairman, and as one of the audience it was a pleasure, to propose a vote of thanks to the Authors of the four important Papers that had been read. He regretted that only one Author was present, but fortunately that was Mr. Clerke, the Author of the Paper referring to the most important of the works described. In that room the audience was drawn chiefly from Westminster and its vicinity, and a Paper of that description was very valuable in reminding engineers that their work was not bounded by the limited horizon of Westminster, where things were perhaps not as bright as they might be, but that the field of engineering was really only limited by the extent of the globe. Unfortunately, engineers were doing all in their power to reduce the apparent size of the globe by making communication so easy, and were covering it with engineering works, so that there appeared to be very little, at the first glance, left for succeeding generations to do; and perhaps nothing would be more popular in Westminster than some great convulsion of nature, which would make less sea and more land by elevating the bottom of the Atlantic. The Papers before the meeting referred to great undertakings in remote parts of the world, and he hoped that the discussion would be worthy of the occasion. At the first glance, those who were not familiar with works carried out in India might fail to appreciate their magnitude; but the first thing that gave him a scale to measure their importance was the quantity of water which had to be provided for in the waste-weir of the Tansa Dam. If he had heard it correctly, it was certainly more than the discharge of any river in this country in its wildest flood, and twice as much as the summer flow of the Nile. To carry out a dam, when one had to deal with floods of that kind coming over the unfinished masonry, and scouring out ravines 40 feet deep, formed an undertaking that engineers had not experience of in this country. There were other points of interest in connection with the valuable but less important works described in the other Papers, such as building dams, not of masonry in hydraulic lime or of cement, but of sand founded upon sand. The Institution might perhaps hear something from Mr. Leader Williams, or some of the contractor's engineers, about a somewhat analogous operation, namely, cutting

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the deep trench of the Manchester Ship Canal alongside the Mersey estuary when sand was intervening. It was not making a dam of sand, but it came to the same thing, and something might be told as to the percolation of water in such cases. The last Paper had a rather comprehensive title, "The Design of Masonry Dams," but it seemed to be confined to suggestions for facilitating the calculation of the outlines of a dam on the ordinary hypotheses set forth by French engineers and by Rankine. Here again was ample material for discussion by practical men as to how far the hypotheses upon which the calculations were based were correct or approached correctness. The form of the dam arrived at was based upon the supposition that masonry behaved as an elastic solid. He would be glad to hear Professor Kennedy's views on that, and also those of Professor Unwin, for, having been searching for twenty-five years in his own practice for evidence of the kind, he had not yet succeeded in obtaining it; and it appeared that the Institution was the place to ask for it, as, although writers of text-books might properly suggest formulas, it was for practical men to decide whether they could or could not be accepted. The Tansa dam was composed of one-third mortar and two-thirds stone, and in the formula for its outline it was assumed that the modulus of elasticity was constant throughout. In all the experiments that he had made, he had found that the modulus of elasticity of mortar, instead of being constant, would range perhaps from 200,000 to 1,000,000, and that of masonry from 1,000,000 to 5,000,000, and that it was not constant under varying stresses in the same specimen of material. These things could not be averaged and difficulty avoided in that way. He admitted that if a constant modulus of elasticity were assumed, the problem to be worked out was admirably simple, but under existing circumstances it was extremely difficult, and practical experience rather than mathematical investigation was of service when masonry or earthen dams were in question. He was very glad that the subject had been raised, because there were many practical men who were also experimentalists in the Institution, and they were competent to deal with it, and to bring forward the results of their experiments. Of course, the Germans had done a great deal in that respect also. He was confident that if engineers had acted upon the assumption that the line of thrust must be kept within the "middle third," it would have led to a great deal of waste in many important structures. He would be glad, in the course of the discussion, to hear if there was any tittle of evidence in engineering practice that such a course was

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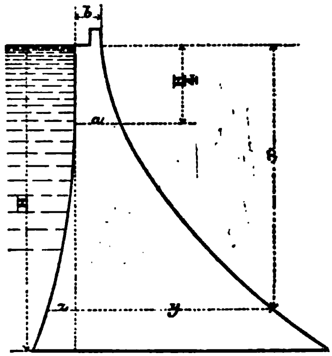
necessary, or whether it was merely an assumption made to facilitate calculation. He hoped that the members would do justice to the importance of the Papers, and he asked them to accord a very hearty vote of thanks to the Authors.

Sir Guilford L. Molesworth.

Sir GUILFORD L. MOLESWORTH was surprised that Prof. Kreuter's Paper contained no allusion to Mr. Bouvier's valuable essay on "Masonry Dams." It was desirable that something about this should be known, because it was shown in it that the formulas of Messrs. Graeff and Delocre, though valuable as far as they went, and exceedingly creditable to those gentlemen in their new departure in high masonry dams, were not complete, but required some modification before they could be considered to give safe results. Before 1866, the factors which influenced the dimensions of large masonry dams were somewhat imperfectly known. The result was that there were many failures of high masonry dams, notably those in Spain, not from the want of sufficient material, but from the way in which the material was disposed. Those dams were crushed by the enormous weight of masonry injudiciously placed. In 1866, Messrs. Graeff and Delocre read an exceedingly important Paper on the subject, and constructed some large dams which quite startled the engineering world with their boldness. The calculations involved in their system were very elaborate, necessitating the use of the integral calculus. He had been asked in 1873 to devise some simpler formulas which would give an approximation to the results obtained by Messrs. Graeff and Delocre, and had undertaken the investigation. At the outset he had discovered a circumstance which had much simplified his labours, namely, that the weight of the masonry did not materially affect the profile of the dam, because though heavier masonry generally involved increased pressures on the base of the dam, yet the resultant of the pressures was thrown further away from the outer face and there was not a greater maximum pressure on the wall. He had therefore prepared a simple formula, which gave results almost in exact accordance with Messrs. Graeff and Delocre's method, though during his investigation he had come to the conclusion that their method was deficient. It gave not the maximum pressure, but the vertical components of the pressure distributed over the surface. That point, or something like it, had been noticed by Professor Rankine, who had said that, in addition to the conditions established by Messrs. Graeff and Delocre, it was necessary that where the wall had a batter, the intensity of pressure at the faces should be diminished below the limit answering to vertical faces. Finding this, Sir

Guilford Molesworth had introduced into the formula a factor which allowed for the obliquity of the face, and published this in the Roorkee professional papers on Indian engineering in 1873. Two years afterwards Mr. Bouvier had published his essay in the "Annales des Ponts et Chaussées," in which, though proceeding on a different line of reasoning, he had arrived at very nearly the same results as those attained by Sir Guilford Molesworth's simpler formula. Sir Guilford Molesworth's original formula was not quite the same as that now indicated on the diagrams, for instance, it would give near the toe of the dam slightly more strength, and about one-fourth of the height from the base slightly less strength. After reading Mr. Bouvier's Paper, with which he perfectly agreed, he had modified his formula slightly so as to be quite in accordance with the method therein set forth. He had adopted the following formula, which was applicable under all sorts of different conditions:— Sir Guilford L. Molesworth.

Fig. 1.



$$y = \sqrt{\frac{0 \cdot 05\, x^3}{\lambda + (0 \cdot 03\, x)}}; \quad z = \left(\frac{0 \cdot 09\, x}{\lambda}\right)^4;$$

Width of top of dam $= 0 \cdot 4\, y$, at $\frac{1}{4}$ H.

Where, *Fig. 1*, H = Total height of dam in feet.

λ = Limit of pressure allowed on the masonry in tons per square foot.

x = Depth below the surface of the water in feet.

y = Offset in feet to the outer face of the dam from a vertical line $= 0 \cdot 6\, x$ as a minimum.

z = Offset to the inner face.

In the diagrams of comparison between Molesworth's formula and Bouvier's method, *Figs. 2*, the limit of pressure allowed was assumed at $5\frac{1}{2}$, $7\frac{1}{3}$, 9, and 11 tons respectively. It appeared from the diagrams that there was an almost exact agreement between his and Bouvier's methods. The difference was small in most cases; it was scarcely visible in the outer face, and only differed materially in the inner face under conditions which only

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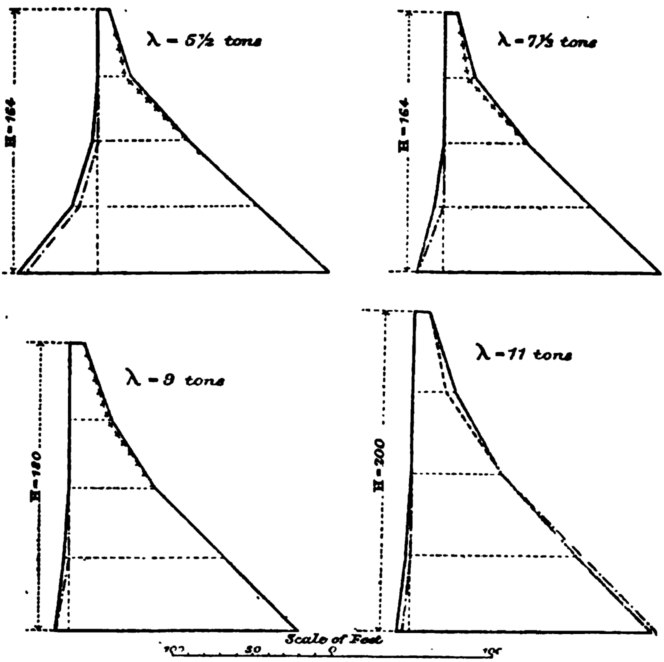
came into action when the dam was empty. The different densities of masonry on which he had calculated those lines were 135 lbs. per square foot; and 150 lbs. per square foot, which was about the limits of practice. *Figs. 3* showed sections comparing his formula with actual practice. The dam constructed at Ban, in France, was originally calculated at 7 tons, but the actual

Figs. 2.



COMPARISON BETWEEN MOLESWORTH'S FORMULA AND BOUVIER'S METHOD.

Molesworth's profile shown thus ....

Bouvier's " Masonry 135 lbs. per cube foot ....

Do. " " 150 " " " " .... + + + + +

Bouvier's profile is not shewn where the divergence from Molesworth's profile is inappreciable.

pressure that came upon it was 9·14 tons. It would be seen that, the profile having been calculated for the lower pressure, the inner face was splayed out more than was necessary. One of the newer dams, at La Terrasse, had been built nearly in accordance with Bouvier's method. The close agreement between the two would be observed. He was not quite sure of the pressure for which the dam at Villar, in Spain, had been calculated, but 6·5 tons was the maximum pressure that came upon it, closely according

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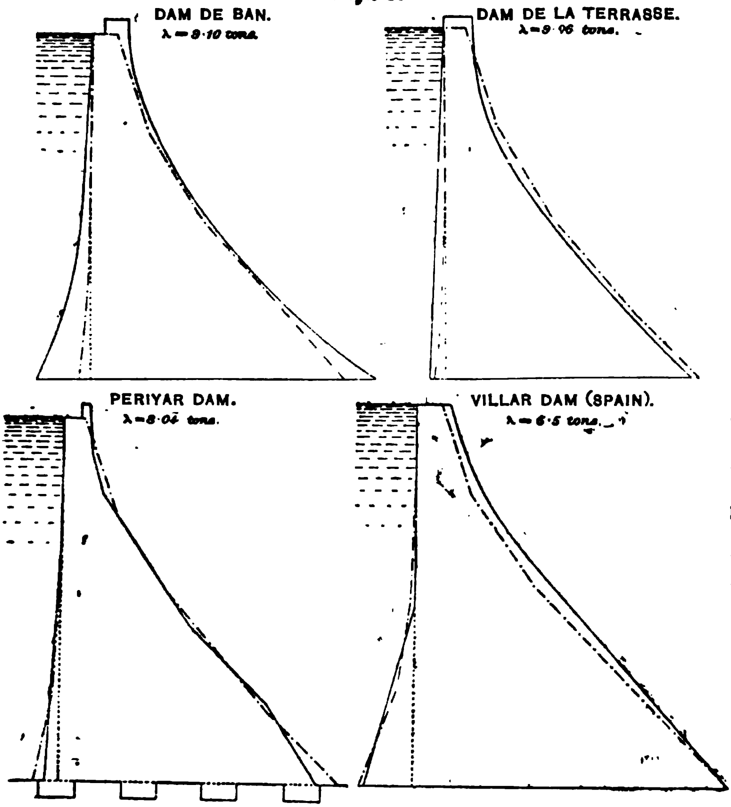
with the line of his formula shown by the dotted line. At Periyar, in India, the dam had been calculated on a basis differing from Bouvier's method, but it agreed closely with it, except at the bottom. He had stated that the diagram showed a comparison between his formula and the actual construction; but in fact the question was referred by the Government of India to

Figs. 3.



Scale $\frac{1}{1,000}$.

him for an opinion, and he had advised that it should be carried out to the dotted profiles representing Bouvier's method, in order to obtain increased safety. The diagram of the Tansa dam showed also a very close agreement with his formula.

With regard to materials, a dam of that sort was always made in rubble masonry, avoiding joints through which the water might percolate. It was really little more than concrete, and con-

Sir Guilford L. Molesworth. sequently the mortar was the factor which determined the limiting pressure which might be put upon those dams. In India the mortar, obtained from kunkur, was of good character, indeed almost equal to cement. It might be said that, so far as Messrs. Græff and Delocre were concerned, those dams which they had put up had stood the test of experience; but they were calculated for a very low limit of pressure; for Mr. Bouvier having made some experiments with mortar and masonry made with the material, had stated that the masonry might well stand a pressure of 9 or 10 tons, and that after it had set a considerable time, it might even be subjected to a pressure of more than 12 tons to the square foot. Some of the earlier dams, the Furens dam, for instance, were calculated for rather less than 6 tons per square foot. Some allusion had been made to graphical statics. He had found graphical statics very useful in determining the size of the dam corresponding with Bouvier's method, and had worked out a system by graphical statics, as shown below.[1]

Determination of Centres of Gravity and Pressure by Graphical Statics.

Calculate the values of z and y as determined by Molesworth's Formulas, and use the figure thus obtained as a basis for determining the centres of gravity and pressure. It is sufficient to divide the section into four imaginary planes, a, b, c, and the base d, then—

1st. To determine the centre of gravity of the masonry in each section separately.

Let A, B, C, D (*Fig. 4b*) be any section (in this case the section overlying the plane d) (*Fig. 4a*).

Draw the diagonals A D, C B; S being the point of their intersection. Make D E = A S, join E C, and bisect the lines C B and E C at F and H respectively; join F E and H B, then the intersection of the lines F E and H B at G gives the position of the centre of gravity of the masonry alone.

2nd. To determine the position of the centre of gravity of the masonry and of the vertical pressure of the water for each section separately.

In any convenient position make a diagram for the polygon of forces (*Fig. 4c*) as follows:—

On a vertical line lay off with any convenient scale, J K = the vertical water-pressure on the face A C, and with the same scale lay off K L = the weight of the masonry in the section A B C D. Take any convenient point M (the position of M is immaterial); join L M, K M, and J M; then these lines give the direction of the lines for the polygon of forces (*Fig. 4d*), which is formed as follows:—

Bisect A C in T, and draw vertical lines through T and G; then from any convenient point N in the vertical line which passes through T, draw the line N R parallel to M K; then from N and R, draw the lines N Q and R Q parallel to M J and M L respectively; the point Q at the intersection of the lines N Q and R Q gives the position of a vertical line which will pass through the mean centre of gravity of the masonry and the vertical water-pressure.

[1] This Paper was published in the Roorkee professional papers in 1883, and a copy of it is in the Library Inst. C.E.

Sir Guilford L. Molesworth.

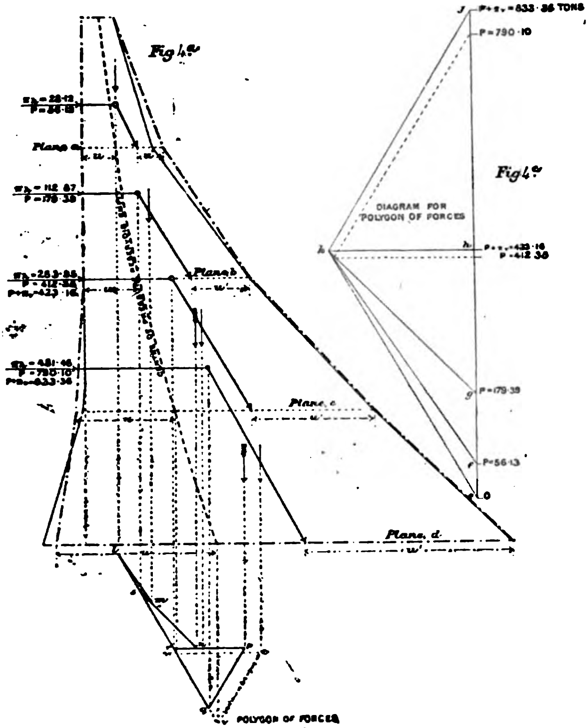


Fig 4f

Fig 4b

Profile determined by Molesworth's formula denoted thus
" " *" Bouvier's method*
Centre of pressure
" " Gravity of Masonry of each Section
" " " " " and Vertical water pressure on each Section
Centre of Gravity of Masonry and water pressure above each plane
Resultant of pressure of Masonry and water

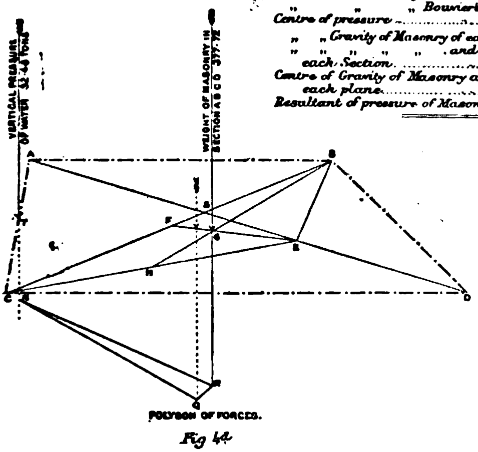


Fig 4d

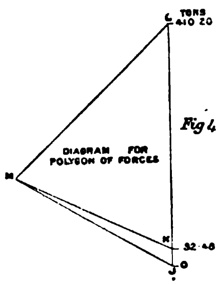


Fig 4c

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3rd. To determine the centre of gravity of the total pressure overlying each plane.

(*a*) *Reservoir full.*—The centre of gravity of each section having been found, as described above, form a diagram for the polygon of forces as before, laying off $ef = P_a$; $eg = P_b$; $eh = (P + \pi_v)_c$; $ej = (P + \pi_v)_d$ (see *Fig. 4e*), P_a, P_b, &c., representing the pressure on the planes a, b, &c., respectively. Take any convenient point k, and join ek, fk, gk, hk, and jk. Then form a polygon of forces (see *Fig. 4f*) as follows:—

From any convenient point l in the vertical line that passes through the centre of gravity of the weights of masonry overlying the plane a, draw lq parallel to ek, lm parallel to kf until it intersects at m, the vertical line which passes through the centre of gravity of the masonry contained between the planes a and b, then through m draw mn parallel to kg until it intersects at n, the vertical line which passes through the centre of gravity of the weight of masonry and vertical water-pressure of that section of the dam that lies between the planes b and c. Then through n draw np parallel to kh until it intersects, at p, the vertical line which passes through the centre of gravity of the weight of masonry and of vertical water-pressure of the section that lies between the planes c and d, and from p draw a line parallel to kj, until it intersects the line lq, then the points of intersection of these lines at s, r, and q, give the position of vertical lines which will pass through the centre of gravity of the loads overlying the planes b, c, and d respectively; the centre of gravity of the weight overlying the plane a having previously been determined.

(*b*) *Reservoir empty.*—The centres of gravity for the empty reservoir are found in the same manner as above, laying off in the diagram for the polygon of forces P, in every case, instead of $P + \pi_v$. The diagram and the polygon of forces in the case of an empty reservoir being shown in dotted lines when they differ from those of the full reservoir.

4th. To determine the resultants of pressure for each plane, the reservoir being full.

Find the centre of horizontal pressure of water for each plane, equal two-thirds of the depth of the plane below the surface of the water, then from each point in the polygon of forces, l, s, r, and q, draw vertical lines to the intersection of the horizontal line of the centres of pressure respectively (these intersections are shown by the centres of small circles in *Fig. 4a*); then, from these points of intersection, lay off with any convenient scale a vertical distance $= P + \pi_v$, and, from the vertical distance so laid off, lay off a horizontal distance $= \pi_h$; then a line from the intersection of the centres of gravity and horizontal pressure of water to the point given by this horizontal distance will represent the direction of the resultant of the weight of the masonry and of the water-pressure, and where this line intersects the plane will be the centre of pressure of the resultant on the plane in question.

5th. To determine the centre of pressure for each plane, the reservoir being empty.

Vertical lines drawn from points v, w, s, l, in the polygon of forces for the empty reservoir to the planes, d, c, b, and a respectively, give points in those planes for the centre of pressure.

The lengths u and u' may then be measured off on each plane from the points found, as described above, in steps 4th and 5th for full, or empty, reservoir respectively, to give Bouvier's profile.

$$u = \frac{2\,P}{3\,\lambda} \text{ when } u < \frac{L}{3}; \qquad = \frac{L}{3}\left(2 - \frac{\lambda}{\left(\frac{2\,P}{L}\right)}\right) \text{ when } u > \frac{L}{3}.$$

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$$u' = \frac{2}{3}\,\frac{P + \pi_v + \frac{\pi_h^2}{P + \pi_v}}{\lambda} \text{ when } u' < \frac{L}{3};$$

$$= \frac{L}{3}\left[2 - \frac{\lambda}{\frac{2}{L}\left(P + \pi_v + \frac{\pi_h^2}{P + \pi_v}\right)}\right].$$

when $u' > \frac{L}{3}$.

Where L = the length of the plane *a*, *b*, *c*, or *d*;
λ = the limit of pressure allowed on the masonry;
P = the total weight of masonry above each plane;
π_v = the vertical component of the water-pressure above each plane;
and π_h = the horizontal component of the water-pressure.

In case the section of the dam is drawn to a small scale, it may be convenient to draw out a polygon of forces to an exaggerated horizontal scale, and transfer the measurement from the polygon of forces to the section of the dam by means of a different scale. Every measurement being reduced to the standard of distance from the vertical line from which the ordinates x and y are measured, and this will save any chance of inaccuracy in projecting the vertical lines from the polygon of forces to the centres of pressure; but for the sake of clearness the polygon (*Fig. 4f*) has been shown directly beneath the sections to which it refers. In the example given the total height of the dam has been assumed to be 180 feet; the limit of pressure λ = 20,000 lbs. (8·93 tons) per square foot, and the density of the masonry = 145·6 lbs. per cubic foot.

Some allusion had been made in one of the Papers to the question of weeping. In one case of a bank impounding water he had found the toe of an earthen bank weeping, and it had caused him some little alarm. He had the water stirred up, mud and clay thrown into it, and kept for some days in a state of agitation inside the dam. In a few days the particles of clay thrown in were drawn in by the leakage, and the dam very soon puddled itself, and the weeping ceased.

In Ceylon impounding tanks from 15 to 30 feet deep were very numerous. There were certain small village tanks which gave a great deal of trouble. The practice of the villagers was to cut the embankments of those tanks to let the water gradually out. They were very skilful in doing it; but it frequently happened that ultimately the water overpowered them and swept away a large part of the dam. As the tanks were very often in series, one below the other, the bursting of a dam was a very serious thing, because the failure extended through the chain of tanks. He had designed a little gridiron sluice, cast on a pipe, which could be bedded in rubble concrete. The rubble was built across a channel, not in the embankment, but in the hard ground beyond the end of the embankment. The results were very satisfactory.

Sir Guilford L. Molesworth. Hundreds of those little sluices were put in by the natives, or by the revenue officers, and they saved an immense amount of waste. It was desirable to keep the outlet-sluices away from the embankment altogether; and he had always made a practice, if possible, of cutting a channel in the hard ground at one side, and he had even tunnelled through rock to get the outlet far away from the embankment and in sound ground.

Mr. Russell Aitken. Mr. RUSSELL AITKEN observed that, when engineer to the Municipality of Bombay, he was the first who had surveyed (1866–69) that site on the Tansa river, but did not consider that it afforded a favourable one for a reservoir. He had recommended that a reservoir should be made at Shewla, as near Bombay and at a greater elevation above it, where a lake 6¼ square miles in area could have been formed by a dam 2,000 feet long and 96 feet high. Compared with this the Tansa dam, 8,000 feet long and 120 feet high, formed a water-spread of only 4½ square miles. The area of the Tansa reservoir could, it might be said, be increased to 8 square miles; but Bombay was unlikely for 100 years yet to require more than 60,000,000 gallons per day, the available daily supply being 15,000,000 gallons from Vehar and Toolsee, 20,000,000 gallons from Tansa, and the further 200,000,000 gallons which might be obtained from Tansa when another 4-foot main is laid. Moreover, the pressure now obtaining in the dam was higher than such masonry should be subjected to, and it would be highly dangerous to increase the size of the Tansa lake by raising the dam in the manner that was contemplated.

Mr. Aitken's experience of lime mortar, or chunam, whilst he was engineer of Bombay, during which time he was entrusted with the demolition of dangerous buildings, had been somewhat extensive; but he had never seen anything like good mortar in the Western Presidency. Whilst the Karackvasala dam, near Poona, 90 feet high, was being constructed, in 1868, he had pointed out that the mortar used was not sufficiently good, having regard to the stress which would be imposed upon it. Experience had proved the correctness of his view, as the water had never been impounded in the reservoir to the full height intended. Further, it had been stated by Major-General Sir Andrew Clarke, R.E., Assoc. Inst. C.E., in a report on the Vyrnwy dam, dated 5th May, 1886, that "The deflection of the Karackvasala dam when incomplete, being about 70 feet high under 50 feet head of water. was ascertained to be ½ inch in a length of about 500 feet." He might mention that the late Mr. Froude, about 1863, was the first to point out that in any retaining wall whatever the resultant

Mr. Russell Aitken.

pressure should lie within the middle third; and in connection with the dam alluded to, he had pointed out to the Government that the resultant stress should lie within the middle third, and not within the middle half of the section as stated by the designer of the work. Whilst preparing his report (1868) on the Bombay Waterworks, Mr. Aitken had not proposed a stone dam because he could not depend on mortar, and cement was, at that time, much too expensive to be employed. About 1876, however, when some merchants and others proposed to impound the water at Shewla for power for spinning cotton, he had estimated for a dam to be built with Portland cement, and the estimated cost did not much exceed that for the earthen dam. The Tansa dam was built with the same kind of masonry and chunam as that at Karackvasala. The former dam was, however, higher than the latter by 30 feet. Chunam possessed the property of apparently setting, and subsequently disintegrating, owing probably to the presence of free or unslacked lime. No one who had seen the ruins of the Mhow-ke-Mullee viaduct on the Bhore Ghaut incline of the Great Indian Peninsula Railway, would feel inclined to trust chunam rubble masonry for great pressures, as hardly two stones adhered together—that viaduct had been rebuilt with heavy block-in-course masonry, in courses 2 feet thick at the bottom to 1 foot thick at the top. In Mr. Aitken's design for the reservoir at Shewla, the main was estimated for in mild steel, 4 feet 6 inches in diameter, carried on masonry pillars, so as to be clear of the surface of the ground. He had found that cast-iron pipes buried in the soil rapidly corroded, owing to the presence of bacteria which secreted nitric acid that attacked the iron. He regretted that his plan for a reservoir at Shewla had not been adopted in 1884, as 25,000,000 gallons of water could have been taken to Bombay by a steel main at less cost than the 20,000,000 gallons were conveyed from Tansa, and the water supplied would have been of better quality than that from Tansa. The gathering-ground at Shewla was hard and rocky, without a tree on it, whereas there was much soil over the rock at Tansa, the gathering-ground being a dense jungle with much decaying vegetable matter on it.

Mr. James Mansergh.

Mr. James Mansergh said, during the last year or eighteen months, in his office some scores of sheets of double-elephant had been covered with profiles of masonry dams, and many hundreds of sheets of calculations had been produced. He, therefore, would not attempt to enter on the question of the theory of dams, but proposed to wait and see if anything else was to be learnt on the subject from those who might take part in the discussion. He

Mr. James Mansergh. would like to ask a few practical questions on matters raised in the first Paper. First, with regard to the flood discharge, it had been stated that the drainage area above the Tansa reservoir dam was 52½ square miles, and the Author estimated that the maximum discharge from that area would be 744 cubic feet per second per 1,000 acres. Putting that in simpler figures it would mean 45 cubic feet per minute per acre. During the eight years that observations had been made on the Tansa watershed, it had been found that the highest flood-discharge had been 30 cubic feet per minute per acre. That would be considered in England a very high flood-discharge, and he knew of no record of such a flood except over a very small area of perhaps 4 or 5 square miles. In a Paper on the Nagpur Waterworks,[1] Mr. Binnie had stated that he had gauged a flood which represented 98 per cent. of 2·2 inches of rain falling in 100 minutes, which ran off the watershed in 170 minutes. This was over an area of 6·6 square miles, and was equal to about 46 cubic feet per minute per acre, if evenly distributed over the 170 minutes. Probably Mr. Binnie had not the means of ascertaining the discharge when at its maximum. It was stated in the Paper that he measured the discharge by the rising of the water in the reservoir, and had no weir to measure his absolute maximum upon. With regard to the flood-discharges, which were most important matters in connection with the design of works, he would be glad to learn what was the exact nature of the Nagpur and the Tansa water-shed areas in respect, for instance, of the elevation to which they rose above the dam, the slope of the ground and its nature, whether hard and impervious or pervious, so that a judgment might be formed as to the rapidity with which the water would run off. The Tansa watershed area was large, viz., 52 square miles, and it would be interesting to know, if there were many streams which brought the water rapidly down to the reservoir. The Nagpur area, on the other hand, was small. Of course, it was quite clear that the flood-discharge per acre per minute must decrease as the watershed area increased, and any information which could be contributed on this point by any member present would be most useful to all. He would ask the Author of the first Paper if at the time of the 30 cubic feet flood there was any record of the rainfall or the time in which it fell; also if any heavy flood had gone over the weir of 1,650 feet long since the work was completed, and if so with what result. There was no cross-section through the waste-

[1] Minutes of Proceedings Inst. C.E., vol. xxxix. p. 1 *et seq.*

weir given in the Paper, and it would appear from the shape of the top of the dam, which was perfectly level and without any curve on the outer face, that in times of heavy flood the water must fall clear over with a heavy blow on the ground below. There did not appear to be any work below that weir; probably the fact was that the rock was close to the surface, and the water had been allowed to fall over and scour the soft stuff off the rock. The fall was now about 28 feet, and when the dam was raised to its full height it would be increased to 45 feet; so that the time might come when something would have to be done. On the drawing of the longitudinal section of the dam there were two lines shown underneath the ground. Probably the Author would explain what those lines meant. It was interesting to learn that the dam, which contained over 400,000 cubic yards of material, was built almost entirely of stones averaging not more than ½ cubic foot each, and that mortar made with soft sand formed one-third of the mass. It was clear that the kunkur lime used in making this dam must have been of exceedingly good quality, and that the work was well put together; because the Author had said that in the early stages the water passed over rough and recent work down a 1 to 1 slope, and that the flood went over 6 feet deep without displacing a stone. Then there was a question with regard to the aqueduct. The cut-and-cover and tunnel-work of the aqueduct was formed of a culvert, 7 feet wide and 7·3 feet high, with a flat bottom, vertical sides and a fall of 6 inches in a mile. The Author had stated that experiments showed that the discharging capacity was practically the same as that deduced from Kutter's formula, taking the value of his coefficient for friction as 0·012 or 0·013. The floor of the conduit was flat, and made with concrete rendered smooth with Portland cement. The side walls were of rubble masonry built with kunkur mortar, the joints having been raked out and pointed with Portland cement. It would be instructive to know exactly what was the character of this masonry. Some stone in this country—dressed as is usual for rubble work—only with a hammer, would present an exceedingly rough surface and cause much obstruction to the flow of the water. It might be that the stone used in this aqueduct had a clean smooth fracture at right-angles to the bed, or could be dressed down cheaply so as to make a wall corresponding fairly with good brickwork. In a recent and somewhat similar case about which he had to settle a dispute, photographs were produced which showed very clearly the character of the work; possibly (now the use of the camera was so common) the

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Mr. James Mansergh. Author might have photographs which would explain exactly what the finished sidewalls of his aqueduct were like. Facts as to the discharge of large culverts of that kind, and also of the 48-inch iron-pipes, gave just the sort of information that was wanted in the Minutes of Proceedings, and all members who had the opportunity of making practical experiments of that class would be conferring a favour on their fellows by furnishing the results to the Institution, giving in all cases an exact description of the character and condition of the structure of the conduit, with the cross-sections and gradients. He believed that Mr. Santo Crimp had in London tested some of the large main sewers in varying states of repair, and other members might have had opportunities of making experiments on a practical scale which would be exceedingly useful. On Plate 2, cross-sections were shown of the aqueduct in the open. The depth of water to the springing was 6 feet, and the thickness of the side-wall was 1 foot 9 inches at the top and 3 feet 3 inches at the floor. He would like to know whether, when the conduit was tested running up to the springing-level, those side-walls were water-tight. The Paper on the Baroda Waterworks did not contain much of interest except that the earthen dam had no puddle wall. There were two puddle trenches, one being under the reservoir bank. The second wall was put inside the reservoir some 150 feet from the foot of the inner slope. It was difficult to conceive exactly what its utility was. There was no section across the valley, and no indication of any puddle sheeting between the two trenches; and there seemed to be a weak place where the water could get down just where its pressure would be greatest. It would be interesting to know exactly the reason for the inner trench. In the Jeypore case there was an embankment made of sand standing upon sand, without either puddle trench below ground or puddle wall above, the height being 61 feet. He did not know that in any country he would have ventured to construct such a work, or that he ever had had clients who would have liked to risk their money in such an experiment. Altogether the work was a mystery to him, and the figures with regard to the rainfall and its utilization were also mysteries. The Author had stated that the mean rainfall— he did not say annual, but it was to be presumed annual rainfall —was 24 inches, and that he had anticipated collecting 4 inches off a drainage area of 13 square miles. But as a matter of fact the whole of the water that came into the reservoir was only one-sixtieth of the rainfall, which would be 0·4 inch per annum. One would have thought that in a district of this character it would

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have been desirable to make a really sound, substantial and water-tight reservoir, capable of storing all the water that could be procured. If he read the Paper aright it appeared that even the small quantity of water that got into the reservoir leaked out again, because the Author described how springs occurred at the foot of the nullah, 200 feet or 300 feet below the dam, showing (as he said) how porous the soil was, and, it might be supposed, how porous the dam was, too. Under those circumstances he could not understand how the water ever rose to 24 feet in the reservoir, and how they had supplied 900,000 gallons a day, when only 200,000, which was the product of 0·4 inch per annum on the drainage-area, was procured. There seemed to be a balance of 700,000 gallons a day unaccounted for. He would be glad to have some further information on that point.

Mr. L. F. Vernon-Harcourt.

Mr. L. F. Vernon-Harcourt had, some years before, availed himself of an opportunity of reading Messrs. Graeff and Delocre's Papers upon masonry dams; and believed that those Papers were really the foundation for the design of masonry dams in use at the present time. Those Papers did not merely deal with theory, but a masonry dam had been erected from the formulas then expounded; and he had the opportunity last year of seeing that dam. It was the Furens dam in the Gouffre d'Enfer, for supplying St. Étienne with water. It was rather a difficult work to visit, being out of the way; but he believed it was, if not the highest, very nearly the highest masonry dam in the world. When he saw it, the reservoir was not quite full; but, compared with another foreign masonry dam which he had seen previously, it was remarkably water-tight. There was no leakage of water visible on the outside, although the dam had to sustain a head of water 164 feet in height. In that respect it compared very favourably with the Gileppe dam near Verviers, in Belgium, which he had described in a Paper read before the Institution in 1889.[1] That dam, though very much thicker than the Furens dam, suffered considerable percolation between its joints; and in fact at the bottom it was remarkably wet, showing that the masonry work was not impermeable, and the importance, in such cases, of providing a watertight coating on the upper face of the dam. The Gileppe dam was, indeed, very much thicker than theory would determine, imposing an unnecessary additional pressure on the masonry. He did not know any reason for that, beyond a statement that it would possibly be heightened at some future time; but it did not in any

[1] Minutes of Proceedings Inst. C.E., vol. xcvi. pp. 187-89.

Mr. L. F. Vernon-Harcourt.

way correspond in form with the Furens dam.[1] With regard to the Vyrnwy dam, he had had the curiosity to take out the bottom width as compared with the Furens; and measuring from the water-levels of the two, he had found the bottom width of the Vyrnwy dam was as nearly as possible the same as the Furens dam at the same depth below the water-level. The Vyrnwy dam, however, attained nearly its bottom width at the ground level, 50 feet above its base, which accounted to a considerable extent for the statement that the Vyrnwy dam was much larger in proportionate sectional area than the Tansa dam, as it was much thicker above the bottom than the Furens, the Villar, and other recent masonry dams, with the exception of the Gileppe dam. Probably the cause was that the dam was used as a by-wash, and also on account of a roadway having been made across it, which was perhaps one of the reasons that the Gileppe dam was made as large as it was at the top. With regard to the Tansa dam, it would, of course, compare favourably in sectional area with the Furens dam, because allowance was made for a greater pressure upon the masonry. As stated by Sir Guilford Molesworth, the maximum pressure upon the Furens dam was 6 tons per square foot; but Mr. Bouvier had shown that a pressure of 10 to 12 tons per square foot was admissible; and therefore the dam might be made slighter. With regard to a point in Prof. Kreuter's Paper about the French engineers not having made mention of keeping within the middle third—although they might have omitted to mention it, they certainly, in the designs made by Messrs. Graeff and Delocre and others, kept the lines of resultant pressures, reservoir empty and reservoir full, within the middle third, and greatly within this limit towards the bottom; so that, as far as that was concerned, it did not signify very much if it was not mentioned among the conditions to be fulfilled. In respect to the design of masonry dams, he did not think that much further progress could be achieved, as Prof. Kreuter appeared to expect, for the general theory was based on definite statical pressures capable of a graphical solution; but further investigation was desirable into the precise distribution of the pressures over the masonry, which was a physical problem of considerable complexity, and which had not hitherto been determined.

Prof. W. C. Unwin.

Prof. W. C. Unwin wished, first of all, to say a word or two of criticism on one or two points directly raised. In the first Paper,

[1] Compare Figs. 5 and 6 in the Encyclopædia Britannica, 9th Edition, vol. xxiv. p. 407.

a comparison had been instituted between different forms of masonry dams, and a statement was made about a so-called "Rankine" section, which conveyed a curious misapprehension. According to a Table given in the Paper, a dam of Rankine section 135 feet high, would be 9 per cent. larger in section than the Tansa dam, and a dam of Rankine section 50 feet high would be 43 per cent. larger than the corresponding dam of the Tansa section. He wished to point out what Rankine actually did. There was sent to him from India a design of a dam something like 180 feet in height. He prepared a report on that dam, which report, for accuracy and dexterity of treatment, was almost as good a piece of work as Rankine ever performed. He proposed to modify the section sent him to a section not widely different from it, but of what he thought a better form, and he gave a design for a dam of that height to the section which he preferred. Taking the top piece of that section, it was quite true that the upper 50 feet would be 43 per cent. more in area than the corresponding part of the section of the Tansa dam; but it must be remembered that Rankine explicitly pointed out that that was altogether a wrong method of obtaining the section for a dam of diminished height. It was pointed out by Rankine that in dams of smaller height, taking the top piece of his section of a dam 180 feet high, would lead to a great waste of material, and he gave a rule for reducing the section in such cases. That should be remembered, because otherwise it was implied that Rankine had laid down a universal section, of which slices might be taken as desired. Professor Unwin next desired to remark upon the curious experience recorded by the Author, on cast-iron water-mains laid above ground and exposed to a great range of temperature, which appeared to have stood perfectly well in that unusual position. They had remained unfilled for some time, during which it was possible that the range of temperature was something like 90° F. For that range of temperature there would have been, if the iron pipes had been placed exactly end to end so that there was no play between them, a longitudinal stress in the pipes, due to their expansion, of something like 4 tons per square inch, which was a very large stress indeed. The stress due to expansion in a case of that sort was longitudinal. The stress due to water-pressure was a hoop tension, and there was no certainty that the power of resisting hoop tension was diminished by the longitudinal compressive stress. Besides that, each length of the pipe would only expand 0·07 inch, and, probably, much of that would be taken up by

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Prof. W. C. Unwin. some little crushing or movement of the ends of the pipes. Some eighteen months ago, in the United States, he had seen a remarkable main laid by Mr. Herschel for supplying water to Jersey City.[1] That was a 48-inch steel main 21 miles in length riveted up from end to end. It was not exposed above the ground as the Tansa main was, but no doubt it must be subject to considerable stresses due to alterations of temperature. So far as he knew that main had suffered no damage that could be attributed to changes of length. In the interesting Paper by Professor Kreuter, which did render part of the designing of dams a little simpler, attention had been called again to Rankine's papers. It was said that Rankine had imposed two new conditions in the design of dams, but it was not clearly distinguished in the account of the conditions to which attention must be given in designing dams, which those conditions were. One was the condition mentioned as No. 3, which was stated in such a way, though no doubt unintentionally, as to convey a wrong impression. It was given thus:—"That at those parts of the profile where the wall has a batter, the intensity of pressure at the faces shall be diminished below the limits answering to vertical faces." It had not been intended by Rankine that the intensity of stress on the oblique down-stream face should be less than that on the vertical up-stream face. It was meant that the intensity of the stress on the down-stream should be the same as on the up-stream face; but Rankine had pointed out that the stress calculated by ordinary methods was only a vertical component of the resultant stress, which was certainly greater, though in a ratio not exactly calculable. There was a limit which the ratio of the oblique to the vertical stress could not pass, and Rankine took an empirical mean between the vertical stress and the maximum possible intensity of the oblique stress. The method of Mr. Bouvier had been once or twice referred to. In fact, it was merely another method of solving the problem of finding the difference between the oblique and the vertical stress.

Coming to the general question of the theory of dams, down to twenty-five years ago all masonry dams which had been built, and high masonry dams had been built for several centuries, were trapezoidal in section. Some of those dams had stood, and some had failed. They had been designed to fulfil the condition that their moment of resistance to overturning should be two, three or four times the resultant moment of the forces tending to overturn

[1] Minutes of Proceedings Inst. C.E., vol. cxiv. p. 418.

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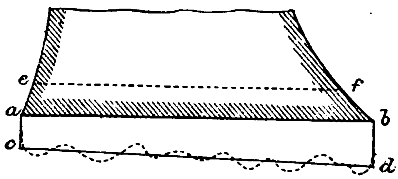
them. He understood perfectly what was meant by the overturning of a block of wood on a table. The forces in that case were so small that the block could be overturned without any sensible crushing of the edge on which it turned. But it was obvious that a high masonry dam could not overturn. For a dam to overturn, the resultant pressure at the face would have to pass through the edge of the base; and long before the resultant pressure could reach that point, the toe of the dam would begin to crush.[1] The French engineers said, "We had better get rid of a theory which assumes that the dam does what is impossible, and we had better investigate the crushing-pressure on the toe of the dam." The whole of their theory was merely the application of mechanical principles to determine a form of dam in which the crushing-pressures should not exceed proper limits on the downstream face when the reservoir was full, and on the upstream face when the reservoir was empty. They arrived at a form of dam much more economical of material than the trapezoidal form previously adopted. For clearness the modern form might be called the "rational section." The rational section required less than half the masonry of a dam of the trapezoidal section previously constructed with the same limits of stress. Since Messrs. Graeff and Delocre had investigated the problem, at least twenty high masonry dams had been built, and amongst those there were only two, as far as he knew, in which the rational form had been widely departed from. The first of those was the Gileppe dam already mentioned, which had a section at least two-and-a-half times as large as it need have had, if it had been built of the rational section. The other was the Vyrnwy dam, which also had a section considerably greater than a purely rational section. He had no doubt that reasons could be assigned for the departure from the rational section in those cases; he was not criticising those dams, but only pointed out that there was no section of any existing high masonry gravity dam which, on the ground of economy, was superior to the rational section. Rankine noticed that many brickwork and masonry structures gave way first at a point where there was tension. Looking to the danger of fissures in a structure supporting water, he considered it advisable that there should be no tension at any section of a masonry dam, when the stresses were

[1] Of course the moment of resistance to overturning about the edge of any horizontal section must exceed the overturning moment. That might be true, and yet the dam might give way by crushing. The ordinary conditions laid down as to the position of the resultant at any joint provided adequate security against overturning.

Prof. W. C. Unwin. determined in the ordinary way. To secure that, it was necessary and sufficient that the resultant thrust at every horizontal section should lie in the middle third of the thickness. This condition had not been attended to by French writers, nor was it satisfied in some French dams. But it was not widely departed from in those dams. He hoped it was not supposed that Rankine ever meant that rule to be adopted as universally applicable to masonry structures; he had indeed given other rules, for instance, for retaining walls, in which he admitted that the resultant pressure at the horizontal joint might be anywhere within the middle three-quarters of the thickness.

The whole theory no doubt depended on treating the high masonry dam as in some sense an elastic structure; but it would not be correct to say that the theory in any way assumed that it was a structure of uniform elasticity. There might be a structure of very discrete elasticity, in which the calculation of stress made according to Rankine's method would be exactly right. In a case of horizontal layers of materials of different elasticity, the stress at the bottom would be exactly that calculated by Rankine. In an extreme case, supposing that the masons put all the pieces of low elasticity on one side of the dam, and all the pieces of high elasticity on the other side, and had a uniform gradation between the two faces, and if there were a variation in the modulus of elasticity of 2 to 1, increase of stress on the base would still not be so great as not to be covered many times over by such factors of safety as were allowed in mason work. There was actually no dam as bad as that. A masonry structure was one of discrete elasticity, but the portions in which the elasticity varied were small compared with the entire mass. According to strict theory, treating the dam as a homogeneous and uniformly elastic mass, the stress on any horizontal section would be distributed as shown at *a*, *b*, *c*, *d*, *Fig. 5*, the vertical ordinates representing stresses. If a dam of discrete material were made up of confusedly arranged pieces of material having different elasticities, there would be more or less variation of the kind shown by the wavy dotted line, *Fig. 5*, but it would be a variation quite covered by ordinary factors of safety. In considering how far the discreteness of elasticity of the stone affected the question of the stress on the base,

Fig. 5.



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only a small thickness above the section considered, for instance, the layer *a*, *e*, *f*, *b*, had to be dealt with. Considering an analogous case, such as that presented by the eye-bar, *Fig. 6*—near and round the pin, the stress in the material was distributed with great irregularity, but the variation of stress disappeared at *a*, *b*, no great distance away. Or considering a block of india-rubber supporting weights, *Fig. 7*—in the layers nearest to the weights the stress was very variable, but at the base of the block it was nearly uniform. In such a mass as that shown in *Fig. 8*, consisting of blocks of stone of irregular shape with intervening mortar of irregular thickness, having a coefficient of elasticity much less than that of stone, in consequence of the varying thickness of the mortar the weight of the upper block would be transmitted to the lower block in such a way that the pressure would be most intense on those parts of the block over which the mortar was thinnest. Consequently, in

Fig. 6.

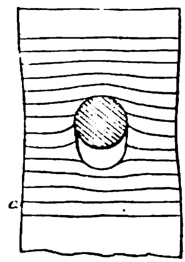


Fig. 7.

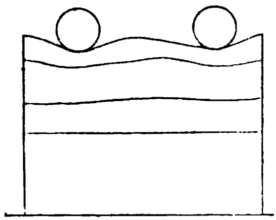
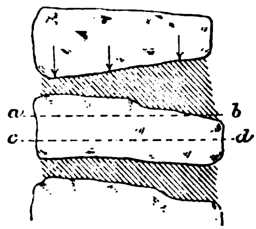


Fig. 8.



the upper layer of the stone *a*, *b*, there would be an irregular distribution of stress. But the irregularity would be less at the section *c*, *d*, being to a great extent damped out, as in the previous cases. The irregularities of stress due to varying elasticity of stones and mortar, were not cumulative from the top of a dam downwards; and there was further compensation from the averaging due to unsystematic distribution of the materials. It followed that at any horizontal layer of a dam, the variations of stress were only those due to a comparatively thin layer immediately above it, and were independent of variations of stress in layers situated considerably higher. Considering the dam as made up of a series of horizontal layers of discrete material, there would be irregularities of stress in every layer, but those would be damped out before they travelled to layers situated considerably below them.

The coefficient of elasticity of stone had not been frequently

Prof. W. C. Unwin. measured. The measurement was extremely difficult, and, apart from observational errors, it was not certain that the hammering to which test-blocks were subjected in cutting them to shape did not affect their elasticity. In the results of Bauschinger's experiments, which were the most careful and extensive that had been made, the modulus varied in the ratio of 10 to 1 when the strongest crystalline rocks and the weakest sedimentary rocks were compared. He was not, however, disposed to think that in the case of rock from the same quarry, the variation of the coefficient of elasticity was nearly so great as that. Further, the general, though not universal, law in stone—that the modulus decreased at first with increase of stress—would have the effect of making the variations of stress less than they would be in a dam of material which had a constant modulus. Should it then be concluded that the discreteness of the dam was of no importance whatever? That would be an error of another kind. He did not think it produced a very great effect on the distribution of stress at the bottom horizontal layer, so long as the stresses on that layer were less than 8 or 10 tons per square foot. But the effect of the discreteness of the material in the dam was that owing to irregular intensities of stress, the average stress at which splitting began at some portion of the interior was much lower than the ultimate resistance of the stone. No one would venture to put on a dam a stress as great as the crushing-stress of the stone determined by experiment. What Rankine did to determine the safe average stress was to examine what average stress was safely carried by existing dams. By determining in that way the safe working average stress, the irregularity due to variation of elasticity of the materials of the dam was, in fact, allowed for.

The examination of the economical form of a dam as a uniformly elastic mass must precede any attempt to take into account subordinate actions due to heterogeneousness. And, as a matter of fact, dams built of "rational section" had proved safe, and were the most economical dams hitherto constructed. Perhaps he might mention that the original limits of 6 tons and 8 tons per square foot for the maximum stress in masonry dams was obtained by examining some of the older trapezoidal dams, which had been built with rather weak mortar. With a very strong mortar, like that used in the Vyrnwy dam, he did not see why considerably higher limits of crushing stress should not be adopted; and in the great dam which was to be built at Quaker bridge for a reservoir for the supply of New York, he understood that a crushing-stress of something like 16 tons per square foot had been adopted.

Mr. G. J. SYMONS said there was a little confusion in the Papers between the amount of loss and of evaporation. In the first two Papers, reference was made to evaporation. The Authors mentioned the evaporation in India as being 72 inches, that was, the evaporation measured from the surface of the reservoirs. He had been pleading for a very long time for observations on evaporation from tanks from which there was absolutely no leakage, and from which positive results could be obtained. Work of that sort had been done by French engineers half-a-century ago; with that exception and experiments at a few of the observatories in Australia, and one or two laboratories in this country—that of the Southampton Waterworks among them—there were scarcely any trustworthy observations as to the loss of water by evaporation from the surface. He did not regard the lowering of a reservoir as the measure of evaporation. He could not believe that any reservoir was so watertight that no water was lost from it otherwise than into the sky. Therefore he did think it was essential to have an accurate knowledge of the quantity of water actually carried away in this country. The Author of the first Paper had stated that he had taken the average annual rainfall at 102 inches. Then he had said he had taken one-third of that as the available quantity. The question was, was he entitled to assume that he could get 34 inches? Certainly if 72 inches were taken off in the first instance for evaporation, that brought the 102 inches down to 30 inches at once; but inasmuch as it was said that 102 was the average, and was not the mean of three dry years, there was a correction to apply, and the result was that nothing like 34 inches would be got. With respect to the tremendous flood-discharges to which Mr. Mansergh had drawn attention, there was a table given of the daily rainfall at the site of the dam at Tansa, from which it would be found that the number of cases of upwards of 4 inches of rainfall in a single day was remarkable—indeed, there were many falls exceeding 5 inches. Of course that was a fall such as rarely occurred in this country, except over limited areas; and therefore was some reason for flows beyond anything experienced in Great Britain. In the Paper on the Baroda Waterworks, the Author had given the rainfall of five years, and the "rainfall discharged." The mean fall was 32·78 inches. The mean discharge was just about 20 per cent. That meant about 6½ inches, which only left a loss of 26·28 inches. That was a very different figure from the 72 inches mentioned in the preceding Paper; and he ventured to think that discordance enough to justify his strong plea for

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Mr. G. J. Symons. accurate observations on the amount of loss from water-surfaces in India.

Sir Benjamin Baker. Sir BENJAMIN BAKER, K.C.M.G., Vice-President, agreed with a great deal that Professor Unwin had said, but some of his assumptions as to the comparative uniformity of the modulus of elasticity were not borne out by the experiments which Sir B. Baker had consulted, and others which he had made. Experiments with limestone and granite at St. Louis Bridge showed a variation in the modulus of nearly three to one in the same stone; and although part of this no doubt was due to imperfections in the mode of experimenting, the inequalities arising in actual structures from imperfect bedding of the stones in mortar of varying consistency, and the greater irregularity of large blocks as compared with carefully selected test-pieces an inch or two in diameter, were probably greater. Very elaborate experiments had been made this year in Germany, by Dr. Hartig, with neat Portland cement in tension and compression at different loads from zero up to breaking; also with three to one Portland cement mortar. He was astonished to find the three to one Portland cement and sand in compression gave a modulus at ordinary working stresses two-thirds higher than the neat cement, the respective values being 2·3, and 3·8 millions, because that was contrary to his own experiments, which showed a large reduction in the modulus, as the proportion of cement was decreased; and this further impressed upon him the great variation which there necessarily was in the elastic stresses on masonry structures from slight and undetectable differences in the cement and sand used for the mortar quite apart from the question of stone. He agreed with Professor Unwin that the form of dam proposed by the French or by Rankine, was infinitely better than the trapezoidal form. In appraising the value of the numerical results as to intensity of pressure and absence of tensile stress arrived at on their theories, it must, however, be clearly borne in mind that the assumptions were, that the modulus was uniform in any particular layer of dam, and that it was proportional to the pressure, and neither experiments nor experience justified such assumptions. The modulus was different for 1 ton to a square foot from what it would be for 10 tons per square foot: therefore, even from the mathematical point of view, not a straight-line stress and strain diagram, as in the case of steel, but a curved line, would result; but that really did not much affect the question, nor did the relatively large permanent set which occurred even at low stresses in masonry; since differences in materials and workmanship, and other disturbing

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causes, were of infinitely greater importance. In all those structures it was very important to remember that all the conditions were not taken into account. Take, for instance, the element of temperature. When the sun shone on the sloping face of a dam with comparatively cool water inside it, it was well known that masonry would expand; but it would in most cases be so difficult to provide for expansion that engineers had agreed generally to shut their eyes to the fact. He had kept a bar of concrete 50 feet long for two years, not in a sunny but in a shady position, and had noted the expansions and contractions. Taking the modulus of elasticity of that concrete bar, he found that if it had been confined there would have been a compression of 12 tons per square foot from the effect of temperature alone. Supposing the bar had been of the best masonry instead of 6 to 1 concrete, the pressure would have been about four times that, or say 50 tons per square foot from change of temperature alone. Of course in a dam the mass of masonry inside would ultimately become of uniform temperature when they got in sufficiently deep from the external faces. There were experiments made sixty years ago in America by building up a maximum and minimum thermometer in a wall 5 feet 6 inches thick, leaving it there twelve months, taking it out and seeing what the range of temperature had been, and it was found to have been about 20 degrees. That was sufficient indication that in a mass of masonry, with a uniform temperature inside there was a varying temperature to a depth of some 4 or 5 feet from the outside, and heavy stresses on the external face of the masonry necessarily resulted. Therefore, when one professed to calculate maximum pressures on the masonry of a dam to decimals of a ton per square foot, it was necessary to remember that was on the assumption not only of perfect elasticity, but also of perfectly uniform temperature. If they took account of the change of temperature they might have local pressures of four or five times that amount; instead of 8·1 tons it might be 40 tons. Occasionally it was found that there were cracks on the surface from that cause alone; for instance, in Colombo the concrete face of a reservoir wall or dam was fissured in all directions, and it became necessary to shield it from the sun's rays by an earthwork slope. The important influence of temperature changes was well illustrated in an experimental masonry arch built in Paris some twenty-five years ago, on which some very valuable experiments were made. Its own weight of about 300 tons, produced a pressure on the masonry of about 20 tons per square foot. It was loaded with 360 tons, which gave an additional 22 tons per square foot,

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making a total of 42 tons. Variations of temperature caused as much difference in the versed sine of the 124-foot-span arch as resulted from 40 tons per square foot variation in stress from loading. It may be remarked further that the modulus of elasticity varied, being at least 80 per cent. higher for the heavy load as compared with the light one, showing that in that case the stress and strain diagram would not be represented by a straight line, but by a curve. Then of course there were those practical considerations that engineers were so familiar with in regard to the chemical action and the consequent expansions and contractions going on in a great mass of masonry or concrete. Many engineers had experience of Portland cement concrete cracking without any pressure on it, by mere chemical action. Masonry in large masses was subject to internal stresses, in the same way as were great castings. The disturbing influences were so many, that although the elastic theory was useful in the hands of practical men in determining the proportions of a masonry dam, he was glad to find that Professor Unwin cautioned young engineers against extending that principle to other structures, where the deductions from it would be wholly at variance with good practice. That had been expressly pointed out by Professor Rankine in the case of retaining walls, but unhappily, by some oversight probably, he had extended it to the case of arches, and had said if the line of pressure did not fall within the middle third the arch would be in a precarious condition. Now, considering that at least 90 per cent. of all the arches in the kingdom were necessarily in that condition it was very important that some protest should be made against such an inference from a theory which even Professor Rankine himself did not accept in other masonry works. Just consider the case of a young engineer who had been taught this doctrine finding the abutment of a bridge or an intermediate pier of a viaduct slightly tilted over after the centres had been struck. He would calculate the consequences of that movement on the usual elastic theory, and find, perhaps, that the line of pressure was not within the middle two-thirds even. Accordingly, if he were acting-engineer of the line, he would possibly think it his duty to stop the traffic, whereas it was known by experience throughout the whole country that there was not the least risk, and trains were running over viaducts in which those conditions existed to an exaggerated extent without any practical inconvenience resulting therefrom.

In the recent experiments of Dr. Hartig on the elasticity of mortars, the result for neat cement at low working-stresses, the

modulus not being constant at different stresses, was 2,800,000, and for 3 to 1 sand-and-cement mortar was 3,800,000. Sir Benjamin Baker's results were, for neat cement, 2,600,000, and for 3 to 1 sand and cement, 1,800,000. That illustrated in an interesting way the influence of sand in the value of the modulus, because the German sand was, no doubt, quite different from the sand that he had used. It was only natural to expect that different sand would vary in the same way as Professor Unwin had stated that different rocks would vary. Certain American experiments made about six years ago with American cement-and-sand briquettes three months old, gave a modulus about one-tenth of Hartig's amount, or 380,000. It might be well to give specific figures by way of illustrating the smallness of the elastic movements, which would follow when they had moduli as high as those to which he had referred. Of course, Sir Guilford Molesworth and every other practical engineer knew exactly what value to attach to decimal parts of a ton per square foot when applied to the calculation of stresses on such a compound structure as a masonry dam, or any other masonry work; but some young engineers and students might not know this, and might attach the same value to the pressures in that case, as they would in interpreting the indicator-diagram of a steam-engine. It was, therefore, as well to point out what those figures meant when applied to stresses of $\frac{1}{10}$ ton per square foot in a dam. The elastic movement corresponding to $\frac{1}{10}$ ton per square foot would be about 1 inch in 40 miles, or 10 inches in the distance from London to Edinburgh. Applying it to a dam 130 feet in height, it would mean $\frac{1}{1,600}$ inch, or just one-eighth the thickness of one of the leaves of the Proceedings. Knowing that fact, and knowing how work was carried out, how stones were bedded by masons, and all the contingencies of different sand and cement and stone, it was known what value might be attached to $\frac{1}{10}$ ton per square foot. Then again, it was known exactly what value might be attached to the bugbear of tension. Assuming an 8 tons per square foot tension, or as much tension as there was compression, in many instances—what, in such a case, would be the opening of a joint if fracture occurred? Supposing the tension extended up the whole height of the 130-foot dam, the collective openings would be $\frac{1}{20}$ inch; supposing that only ten joints opened out of the whole number, it would be $\frac{1}{200}$ inch opening at each, and any slight percolation, of course, would only extend through the face of the work until it reached the first vertical bond where it would be blocked. There was, therefore, not so much to fear as

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regards tensional stresses. He had referred to certain disturbing causes of which engineers had to take account. The elastic movements were very small, but the four disturbing causes to which he had referred were not so: the variation in the modulus, of course, might be very large, as also were the changes of temperature, and the expansion and contraction which took place from the chemical and molecular changes of the mortar during setting; and, finally, the ordinary contingencies of workmanship. Those disturbing causes were really what determined the design of masonry work and the stresses thereon. Practical men knew perfectly well that it was no use for engineers to say they would not have tensile stresses, or a tendency to tensile stress, in their works because it was not for the engineer to say whether he would have it or not. It might be possible in Mars or Saturn, but it could not be true in this planet, because here there had to be met disturbing influences of temperature, chemical changes, and other contingencies. He had referred to his experiments as to the effects of changes of temperature on a 50-foot concrete bar, and, applying the data to a dam of 130 feet, it would give $\frac{3}{8}$ inch as the tendency which the outside layer, and to a lesser extent the layer extending, say, 6 feet in, would have to expand; and that, of course, entirely swamped the elastic stresses when such a high modulus had to be dealt with. Then came the changes which cement mortar underwent in setting according to the quantity of moisture present, the quality of the cement, and other varying elements. Thousands of experiments had been made, and many of them were no doubt recorded in the Transactions, of the expansions and contractions of cements and concrete. Numerous other experiments had been carried out in Germany and America; and it would be found, on looking at these, that there might be as much change in the thickness of a single mortar joint as $\frac{1}{10}$ ton per square foot would cause in the whole height of that 130-foot dam. The most convenient and practical way of obtaining the value of the average modulus was naturally, in the case of masonry arches, by noting the settlement which occurred when the centres were struck, and also the rise and fall of the arches from changes of temperature. In the case of the 124-foot span experimental arch to which he had referred, the rise and fall from changes of temperature was exactly the same as that induced by a varying load, causing a pressure of 30 tons per square foot on the masonry, showing what a very important bearing temperature had. It had been shown by Prof. Unwin how variation of pressure might crush some stones a little,

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but that the average pressure, as calculated, would, he thought, be soon regained. One important thing, however, to consider practically was that in carrying on the building of a great masonry work all the year round, when great masses of the stones put in at one time were necessarily bonded into other masses of work at a different temperature. In the same way the stones were, in practice, not of the same quality, as different quarries and different beds of the same quarry varied enormously. The result was that engineers were bound to accept the fact that, in any great mass of masonry in cement or in mortar which set rigidly as indicated by these moduli, there must be internal tensional stress. If the work was properly designed and executed, the consequent movements were so small that they would not be seen unless the mortar were scraped away and a magnifying glass of high power were used. Then, again, as a rule the engineer was absolutely indifferent as to whether the crack was of that minute size or ten or twenty times as big, because the vertical bond in the masonry, and the care exercised in carrying out the work in good materials, allowed of the tensile stress exceeding the breaking strength, in the same way as the provision of ten to twenty per cent. of elongation in iron and steel rendered harmless the results of all those practical contingencies and variations which could not be evaded in this planet. There was no harm, for example, in young engineers calculating the elastic stresses on a rail by treating it as a continuous girder resting on the sleepers as supports, because, although in practice there might be, say, 20 tons per square inch tension occasionally, where his calculations indicated 7 tons compression, experience had shown that good rails were tough enough to take care of themselves. On analogous grounds there was no harm in calculating the presumed elastic stresses on masonry if the work was strong enough anyhow. Formulas based on average results were very useful to engineers, provided the engineer had in his mind that they were averages, otherwise the same wrong inferences might be drawn as would be drawn with regard to the climate of this country if one went to the Royal Observatory and asked the average temperature of the coldest month in the year, and being told it was thirty-nine degrees, were to assume that water-pipes would never burst nor canals be stopped by ice in this country. It was the deviation from the average which really was so important in the design of engineering works. To sum up his remarks, he would say that masonry dams, or any other engineering works, could not be designed by a formula alone; formulas were only safe in the hands of men who bore in their minds

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the results of practical experience, not only their own but that of others.

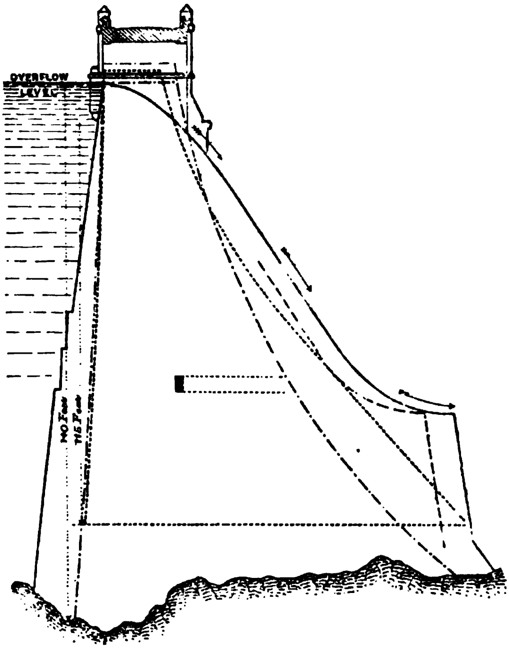
Mr. G. F. Deacon.

Mr. G. F. Deacon said he had been much struck by the high rate of discharge from the Tansa drainage area, amounting to 744 cubic feet per second per 1,000 acres. This for 33,600 acres much exceeded anything which had been recorded in this country. In the Vrynwy valley in Wales, where he had continuous records of the discharge for the last fifteen years, the highest flood from an area of 18,000 acres had been 178 cubic feet per second per 1,000 acres; but he thought it probable that the maximum might prove to be 300 cubic feet. From the drainage area of 6,000 acres above the Furens dam it had been recorded that, before the construction of that work, a discharge of nearly 750 cubic feet per second per 1,000 acres took place, owing to the bursting of a water-spout. He might mention that there was a tributary stream draining nearly 3,000 acres to the Vyrnwy valley, in which a great flood had occurred shortly before he knew the locality. He had made a rough approximation to the discharge on that occasion, and it certainly exceeded 1,000 cubic feet per second per 1,000 acres. At the time that flood occurred, the flow in the main river, draining about 18,000 acres, was very moderate, and he had little doubt that the cause was a water-spout. Such concentrated discharges, known in America as "cloud-bursts," undoubtedly took place, and it was to be regretted that so few exact observations had been made concerning them. They had comparatively little effect on the larger drainage areas employed for waterworks, but they bore importantly on the subject of the proper length of by-washes for smaller areas.

He was obliged to draw attention to an error that occurred in the first Paper, and had also been published elsewhere, with reference to the section of the Vyrnwy dam. The section given was not that which had been executed, and it was moreover so placed in reference to the Tansa section and Professor Rankine's section, as to show the sill of the dam—which acted as a weir—level with the tops of the walls in the other cases, whereas it should obviously have been placed level with the top water in the other cases. These inadvertent errors gave an incorrect idea of the relative volumes of masonry in the different sections. As he believed the section of the Vyrnwy dam as executed had never been published, he showed it in full lines in *Fig. 9*. The original design as signed by the late Mr. Thomas Hawksley and himself, and approved by the Corporation of Liverpool—as shown by the line of long dots—was considerably narrower in the base than

the executed dam. This dimension and the superstructure were afterwards modified for reasons which he need not discuss. The former published drawings had shown the enlarged base, but not the reduced upper portions. From these causes various statements of the sectional area of the Vyrnwy dam and the comparisons between it and other dams, including those made in the Paper, and during the present discussion, had been considerably in error. The short dots in *Fig.* 5, p. 16, showed the section of the Tansa dam Mr. G. F. Deacon.

Fig. 9.



VYRNWY DAM.
Scale, 1 Inch = 50 Feet.

as actually carried out, while the dot and dash lines showed the section of Professor Rankine designed on the hypothesis of uniformly varying vertical pressures at each horizontal layer—an hypothesis which was useful in such work, but which, being untrue, should be employed with great caution. It was important to notice that the horn-like projection or bench on the downstream side of the Vyrnwy dam was not to be taken into account in the area or weight of the structure considered as a gravity dam. That bench was simply for the purpose of discharging in a hori-

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zontal direction along the river-bed the overflow waters which might rise above the sill to a depth of more than 2 feet. In the Vyrnwy dam—the first high dam constructed to act as a weir, thus dispensing with a separate by-wash—some additional thickness was properly given to the apron, though, owing to the roughness of the masonry the stones of which projected intentionally in many cases nearly 1 foot from their draughted edges, the water was entirely broken into spray, and no heavy fall took place at any point. In all masonry dams to be used as weirs that was a necessary precaution. If, moreover, the water carried with it no heavy detritus or stones—which it could not do if the dam were high—there need be no apprehension of serious wearing of the masonry. Another point to be considered in designing the section of any dam was the length along the axis of the dam to which the maximum section applied. In the case of the Tansa dam, that length was probably not more than 200 feet. In the Furens and most other high dams it was considerably less, while in the case of the Vyrnwy dam, the maximum section extended over something like 600 feet in length. Whatever differences of opinion might obtain among engineers as to the proper section for a high dam in a valley of such width that at its centre no effective support was derived from the sides, all must be agreed that in narrow gorges such as those of the Furens in France, the Gorzente in Italy, or the deep portion of the Tansa valley in India, the section might be much modified in figure and reduced in area. No direct conclusions could, therefore, properly be drawn from the experience of dams in such narrow gorges when designing dams to cross wide valleys.

Some differences of construction between the Vyrnwy and the Tansa dams were worth noting. In the case of the Tansa dam, each stone averaged in size half a cubic foot; whilst in the case of the Vyrnwy dam, each stone averaged in size 40 cubic feet or 80 times as large. He was not disposed to think that there was any serious disadvantage in the use of small stones. He doubted whether he should break up stones for the purpose of making them small; but it was certainly more difficult to lay a large stone perfectly than a small one. All things considered, if it were as easy and convenient to use comparatively small stones—not, however, so small as half a cubic foot—he should use them in preference to the large stones, averaging 3½ tons and occasionally 10 tons in weight, which he had employed in the construction of the Vyrnwy dam. Stones of that size could only be properly used with the beds worked, as at the Vyrnwy, to a flat but rough

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surface; and it was necessary to lower them with powerful cranes upon a level bed of fine mortar and to beat them down afterwards with heavy malls. The Author of the first Paper had apologised for having entered into so much detail concerning mortars and other matters, but it appeared to him that in the building of works of that description, almost everything depended upon the details and upon close and intelligent supervision of the work. The stones of the Vyrnwy dam were jointed entirely with Portland-cement mortar; but he was disposed to think that the necessity for the use of such mortar in inland works had been somewhat unduly exaggerated, by reason of the great advantage it had over hydraulic-lime mortar for marine works. He thought that masonry dams might be built, as were several in Europe, quite as satisfactorily as regarded water-tightness and stability, to such heights as had hitherto been attained, with hydraulic lime as with Portland cement. It was worth while to note the actual strength of the materials used in different parts of the dams which had been considered. In the Tansa dam he found that the average ultimate strength, under compression, of the hydraulic mortar employed was 67 tons per square foot. In the case of the Vyrnwy dam, the ultimate strength of the Portland-cement mortar was 285, a ratio of 1 to 4¼. Then came a much more remarkable difference. If the minima in the two cases were compared, the ratio, instead of being 1 to 4¼ fell to 1 to 20, while the ratio between the maxima rose to 1 to 2. Whether that was due to inferior workmanship in some of the samples from the Tansa dam, or to differences in the quality of the lime he could not of course say, but it showed a remarkable irregularity of strength in the Tansa specimens. It had been stated that the hydraulic-lime mortar, investigated by Mr. Bouvier, gave a crushing-strength of 91 tons per square foot. That mortar was made from Theil lime, which had a high reputation; but there were limes in this country which were still better. A material which he had investigated in connection with the construction of masonry dams was pozzuolana, used in conjunction with rich or nearly pure limes. With good natural pozzuolana, such as could be obtained in Italy, the strength of Portland cement could be more nearly approximated to than by any natural lime which he had found. The best concrete he had made with pozzuolana and lime was about half the strength of the strongest Portland-cement concrete; but about 40 per cent. of that considerable strength was obtained by virtue of grinding the pozzuolana very fine in its dry condition. The Italians did not grind the pozzuolana before mixing it with lime, and it was fre-

Mr. G. F. Deacon. quently so mixed when in a coarse condition; by this neglect they lost 40 per cent. of the strength which might be given to it. For a few pence per ton it might be ground to the necessary finenss—ground before being mixed with the lime and afterwards ground again in the mortar-mills with water. With respect to the water face joints of the Tansa dam, which had been raked out to a depth of 1 inch and pointed with Portland cement, his experience had been that work of that class was not of much permanent use. Under the Indian sun, and especially where the work was liable to became alternately wet and dry, he should say that Portland-cement pointing would not last very long. In the case of the Vyrnwy dam, the joints were not raked out, but a space was left without any mortar for 6 inches deep from the face of the wall at the base, reduced to 3 inches near the sill. During construction that space was filled with padded irons which were subsequently withdrawn—by which means the open joints were left perfectly clean, and were afterwards not pointed, but treated with Portland cement and hard pulverized rock in equal proportions, used with very little water and caulked with blunt-nosed punches and hammers. Such caulking was kept $\frac{1}{4}$ inch to $\frac{3}{4}$ inch back from the edge of the stone and was proof against any changes of temperature and perfectly water-tight. For the purpose of allowing the floods to pass over the Tansa dam and of raising the water in front of it during the wet season, it had been raked with a batter of 1 to 1 downwards from the water-face. He scarcely thought that was a course which it would be desirable to imitate. One high dam he had seen after failure; that dam, he believed, had failed by shearing in a plane inclined downwards from the water-face at a slope of nearly 1 to 1. Happily it had not become a ruin, and, having been repaired, was still in use. Perfect continuity of structure was difficult to attain even after the lapse of forty or fifty hours without work—as from Saturday to Monday, when, notwithstanding the greatest care, planes of weakness occurred. A cessation of work from season to season was of course worse. Such planes of weakness were possibly not avoidable, but they should be prevented from occurring in directions approaching those of the principal shearing planes.

Referring to Sir Benjamin Baker's interesting speculations as to the action of temperature on works of this description: When the Vyrnwy dam was in process of construction, certain stones were bored vertically as they were laid one above the other, so that a bore-hole of about $4\frac{1}{2}$ inches in diameter passed from top to bottom near the middle of the work. On the section, *Fig. 9*, was

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shown by a small black rectangle, the cross section of a tunnel which passed longitudinally through the dam rather more than 80 feet below the sill. Into the middle of that tunnel the bore-hole opened, from the top of which was suspended a steel pianoforte-wire carrying at its lower end a very heavy weight immersed in water 80 feet below. That wire was put in in 1887 and was still steel and in existence—a fact attributable to the quantity of lime-water which ran down it. Near the bottom of the wire was an apparatus of the seismograph kind by which any movement was multiplied four times. A fine needle followed the magnified movement upon and along a radius of a horizontal revolving sheet of plate-glass, smoked at the surface. There was no sensible friction, and a zero line could be traced by turning the disk at any time. Unfortunately, no satisfactory records were obtained until the water had reached within 13 feet of the sill, but the movement of the whole dam during the rise from the 13 feet level to the top water-level was such that the sill moved horizontally with respect to a point in the dam 80 feet below to the extent of 0·868 of a millimetre. Earth tremors were clearly recorded, and changes of temperature caused the dam to move sensibly from night to day and from day to night. The dam faced nearly south-east, so that it did not receive the full heat of the sun; but when the reservoir was full, the difference between a hot summer day and a summer night was 0·366 of a millimetre. That was the maximum horizontal movement in reference to a point in the dam 80 feet below—attained some time during the afternoon—of the sill of the dam towards the water, from the place it had occupied on the previous night, due to the expansion of the outer face of the wall.

An interesting fact was given with respect to the discharge of water through the siphons on the Tansa aqueduct, stated to have exceeded 21,500,000 gallons. It seemed at one time to have been calculated by Beardmore's tables (Eytelwein's formula), which gave too low a result for all pipes, more than 3½ inches in diameter; so worked out, the discharge would be only 16,665,000 gallons a day. Then came Darcy, who until lately had been by far the best authority on these subjects, and his formula would give 18,725,000 gallons. More recently Mr. Hamilton Smith had taken immense trouble to collect and formulate the results of various experiments, including his own. Professor Unwin again had investigated the results and had brought matters up to the latest date. Applying to the case in question Professor Unwin's coefficients, he obtained, assuming the pipe to be perfectly clean, a

Mr. G. F. Deacon. discharge of 20,874,000 against 16,600,000 according to Eytelwein; and if, as the Author believed, the aqueduct had discharged 21,500,000 gallons, it somewhat exceeded the highest duty that could have been assigned to it as the result of previous experiments. It would be useful to know a little more about the circumstances under which the observations on the Tansa aqueduct were made.

He had carefully studied Professor Kreuter's communication on the design of masonry dams. Like almost everything that had hitherto been written on the subject, it adopted the methods of statics as applied to rigid bodies, and involved the hypothesis of uniformly varying pressure at every horizontal layer. Those methods were no doubt applicable also with sufficient accuracy to the approximate determination of the stresses in certain simple forms of isotropic solids when subjected to direct compression or extension, but he had pointed out eight years ago[1] that in an isotropic solid, such as a masonry dam, having the section, for example, of a right-angled triangle, subjected only to the forces of its own gravity and the horizontal water-pressure against the perpendicular, that hypothesis could not be true. As the specific gravity of the dam was reduced, the base became longer, until a very acute angle at the outer toe might result; in which case, if the dam were considered as a uniformly elastic solid, it was obvious that the action of gravity and water-pressure alone could not produce a maximum vertical pressure at the outer toe. He ventured to think that a true solution of the problem was a very much more difficult matter than had been assumed. The subject had undoubtedly been treated by Professor Kreuter, on the hypothesis of uniformly varying pressure, with great ingenuity; and he hoped to see his ingenuity and power of analysis applied to the problem of approximately isotropic solids as they occurred in practice. He advisedly said "approximately" isotropic; for a great dam carefully constructed with rubble masonry of uniform quality, though made up of parts of diverse elasticity, was, as a whole, approximately isotropic. Sometimes, however, the faces were built with larger stones and contained less mortar than the interior, and that, no doubt, gave to those faces a modulus of elasticity higher than that of the interior. Although any sudden change in that respect was to be deprecated, it was satisfactory to observe that, so far at least as the outer face was concerned, the increase of the

[1] Report as to the Vyrnwy dam by George F. Deacon, 1885. Pamphlet in the Library Inst. C.E.

modulus in that manner tended to satisfy the hypothesis of uniformly varying vertical pressure when the reservoir was full. It was very far, however, from completely satisfying that hypothesis. He could not leave Professor Kreuter's Paper without referring to his treatment, in connection with the stability of a dam, of the vertical water-pressure over the inner toe. Where, with the primary object of diminishing the vertical pressure upon the masonry at the inner toe, the inner face of the dam was battered, it was undoubtedly correct to take into consideration the effect of the weight of water lying above that batter in reduction of the effect of the horizontal pressure, and this Mr. Deacon had always done. But Professor Kreuter had gone much further than this, and with a view to causing that vertical water-pressure to countervail the horizontal pressure, and thus (again on the hypothesis of uniformly varying vertical pressure at the base) to produce a diagram of nearly equal vertical pressure at different points along the base had proposed to extend the inner toe from the main body of the dam and to terminate it at a very acute angle. For such a purpose he hoped this construction would never be trusted. Masonry, though sometimes apparently water-tight when exposed on one side to the air, was never so absolutely; and even if the rock below the inverted spandril were quite impermeable, which was unlikely to be the case, the masonry could not be so. Adhesion to the rock there might be, but it would only require the element of time to saturate the base with the pressure due to the head of water, and to maintain that pressure until the weight of the overlying water was partly or wholly balanced, and it became of little or no effect in the direction intended. But supposing that projecting toe or spandril, the rock below it, and the cement between the two to be absolutely impermeable, a new element of danger arose. Throughout the investigation Professor Kreuter had evidently assumed that shearing forces and bending moments might be neglected; but Mr. Deacon would invite his attention to the bending moment, assuming impermeability, to which the toe in question would be subject. Impermeability in this sense was, however, out of the question in practice, and the fact was fully recognized in the construction of the Vyrnwy dam. Here the rock foundation was sound, and only in one or two places were there any signs of oozing water; but what pressure might accumulate if that water were held back, especially when the reservoir was full, it was impossible to tell; though it was obvious that, having its origin at higher points, it might even exceed that due to the head of water in the reservoir. The late

Mr. G. F. Deacon.

Mr. G. F. Deacon. Mr. Hawksley, who was pre-eminently qualified to judge by vast experience and acute sense of the numerous difficulties attending such problems, laid great stress upon that point; and Mr. Deacon cordially agreed with him as to the necessity for so connecting any suspicious parts of the rock with the down-stream side of the dam, that under no circumstances could an up-lifting pressure occur. The Vyrnwy rock—clay-slate beds of the Lower Silurian formation—dipped at an angle of about $1\frac{1}{2}$ to 1 towards the reservoir. Along the inner angles of the numerous outcrops at which water, if it penetrated through the rock at all, would issue, drains were laid, and from those drains numerous other drains were brought up through the masonry to the longitudinal tunnel shown in *Fig. 9.* Thus any possibility of up-lift of the kind in question was averted. The Vyrnwy dam was sensibly water-tight; and almost all the water issuing from those drains, therefore, came from the rock which when bared was almost dry. It now amounted, reservoir full, to about one-third of a cubic foot per minute and varied with the depth of the water in the lake. The precaution of draining the foundations he was similarly adopting in other cases.

Mr. Baldwin Latham. Mr. Baldwin Latham, having had the opportunity of seeing the Tansa Works in progress, wished to offer his congratulations to the Author of the Paper on having brought this great work to a successful termination. It was a work reflecting the greatest honour upon Major Tulloch who had originally proposed it, upon the Municipality of Bombay who had found the funds, and upon the Author who had been its engineer. Mr. Latham had been professionally engaged upon the Baroda Works and had made two inspections of them, so that he knew something about these two works. The only fault he could find in the former was in the design of the face of the dam. The Author had said that it was not ashlar work, but it appeared to be something of the kind; it was coursed masonry, and it was like putting a veneer on to very substantial work, which added neither to the beauty nor stability of the dam. With regard to the materials used in the construction of the Tansa Works, in his judgment no material was equal to the kunkur lime which had been used in that dam, and for that purpose it was greatly superior to any Portland cement. Under similar circumstances Portland cement was very liable to failure, but that lime had special qualities to recommend it as vastly superior for work of that class. The fact that the work had been brought to a successful issue was conclusive proof that the materials used had been satisfactory. The stone was the best that could be secured for

Mr. Baldwin Latham.

making the dam water-tight; and, being extremely heavy, strong and impervious to water, gave great stability to the work. At Baroda, there was an earthen dam nearly three miles in length, enclosing a large area of water, collecting the water from a drainage-area exceeding 36 square miles. The ratio of the area of the water of the reservoir to the drainage-area was greater at Baroda than at Tansa, the reservoir-area being over 13 per cent. of the drainage area at Baroda and only about 9·5 per cent. at Tansa. At the latter place there was over 102 inches of rainfall on the average of six years in the monsoon period, and at Baroda in the same six years there was 62·6 inches of rainfall. The result of the large area of the reservoir at Baroda, and the amount of water lost by evaporation and percolation as compared with the rainfall and flow off the ground, gave very different results from those at Tansa. On the average of six years, out of 36·2 inches of rainfall only 3 inches actually flowed off the watershed, against six times that amount secured at Tansa. Curious as it might seem, although the Baroda reservoir would hold a quantity equal to the supply for eight years or more, taking the supply at 4 cubic feet per head per day for a population of 120,000 persons, yet, if the supply had been drawn at that rate from 1886 to 1892, there would have been during three years a short supply of water, owing to the enormous loss from evaporation from the large reservoir-area. The evaporation, absorption and percolation taken at 6 feet upon a full reservoir more than ordinarily used up the supply which could be collected from that particular area; and the consequence was that he had advised the authorities that it was most desirable to bring in another source of water-supply. Another reason for that was that having to deal with an area on which such an enormous amount of evaporation took place, the quality of the water supplied would deteriorate in the reservoir, unless there were an opportunity of an overflow, so that it could be thoroughly flushed out. That was one of the important points he had to consider, and he had recommended it as most desirable that, hereafter, when nearly the full supply was attained, water should be conveyed from another catchment-area into the existing reservoir. When he first inspected this work the dam was not complete, and failures had taken place in some portions of it, entirely due to their having been constructed in the black soil of India. The monsoon descending upon the dry soil had damaged some of the work in various directions. He had then ascertained a very remarkable fact, viz., that it seemed almost impossible to build an ordinary wall which could withstand the stresses due to the expansion of

Mr. Baldwin Latham.

the soil after the dry period in India. The rate of shrinkage in a large number of experiments averaged about 19 per cent., and a soil expanding and contracting at that rate would be destructive to almost any wall which could be put against it. After the dam at Baroda had been closed there appeared to have occurred a large amount of what the Author had termed natural percolation under the dam. A large number of experiments were made to ascertain whether that was really percolation or leakage. Any water passing through the dam must be looked upon as leakage, as distinct from spring-water percolating under the dam. What was thought to be percolation under the dam Mr. Latham was soon able to prove was not percolation but leakage through the body of the dam. The way it was done showed clearly that the law which governed the flow of underground water was similar to that which governed the flow of water in pipes or channels, *i.e.*, if the leakage had taken place underneath the puddle, the apertures by which it was escaping would always be of the same size; and, therefore, the volume of water escaping would always be proportional to the square root of the height of the water above the point at which it was escaping. The volume of water escaping could be determined with accuracy, but when gaugings were made, it was found that the volume escaping bore no relation to the square root of the height of the water in the reservoir above the point, which should have been the case if the water passed from the reservoir under the dam. It was, however, found by taking into consideration another factor, *i.e.*, the actual height of the water in the reservoir above the top of the puddle-wall. That wall had only been carried up about a foot or two above the ground-line, so that in the upper portion of the dam there was no puddle-wall. It was then found that the escaping water could be calculated with exactitude, showing that the escaping water passed through and over the puddle-wall. An examination of a number of samples of material taken from the places from which the soil of the dam was procured, showed that the rate of absorption of water varied between 18 per cent. and 43 per cent., or an average of 31·8 per cent.; and that on the average of all the samples, every cubic foot of that dam would hold 2·7 gallons of water. That being so, and having regard to the enormous quantity of water the reservoir held and the existing leakage, he had recommended the authorities to increase the outer slope of the embankment, so that now, instead of being 2 to 1, it was 3 to 1 at the lower part. As the dam had been built of such materials as were available, and was evidently leaky, and as the only mode of emptying it in case of

accident was by a 30-inch outlet-pipe—bearing in mind that the lake was situated at a point about 12½ miles from the city of Baroda, and if at the time of the monsoon, when the rivers were full, the reservoir were to burst, the serious consequences which would happen to that city—he had strongly recommended the authorities to put in sufficient sluice-area near the side of the overflow where the ground was very suitable, by which means the water could be rapidly lowered in the reservoir without danger. With regard to the construction of the works, he must say he never saw a more perfect work than the slope of that dam. The inner slope was 3 to 1, 3 miles in length, pitched with brick-on-end, and better work it was impossible to imagine. There was not the slightest depression in any part of it. It reflected the greatest credit upon those who were concerned in its construction; and when guarded by such an arrangement as he had suggested, would no doubt be a great and permanent benefit to the city of Baroda. The whole of the work had been done, not by the Municipality, but by His Highness the Gaikwar at his own cost for the public benefit. Mr. Baldwin Latham.

Mr. A. W. Brightmore restricted his remarks to the subject treated in the fourth Paper, viz., the design of masonry dams as distinguished from their construction. There were only two practicable methods of designing a masonry dam. A section might be first drawn, and the stresses in it be subsequently analysed (generally by graphic statics) on the assumption of uniformly varying stress—the profiles being modified until the desired stresses were obtained. In the second method, by the aid of the same assumption, formulas were deduced by which the proportions of the dam might be calculated. Sazilly, Graeff and Delocre were the pioneers of the latter method of investigation, and the Tansa dam, originally proposed by Major Tulloch, was the origin of Rankine's examination of the question, which was the first English investigation. Delocre's method, however, was cumbersome, requiring tentative solution and resulting in an equation of the sixth degree. Rankine and Bouvier had contented themselves by giving expressions for the profiles of a dam, which were only equations to simple curves found to cover the profiles calculated according to Delocre's methods. Rankine, however, introduced the important condition, that there should be no tension in the dam, necessitating the resultant pressures, reservoir full and empty, falling within the middle third of the section. It had been asked, what evidence there was for the introduction of that condition? An answer was, the evidence afforded by experience of Mr. A. W. Brightmore.

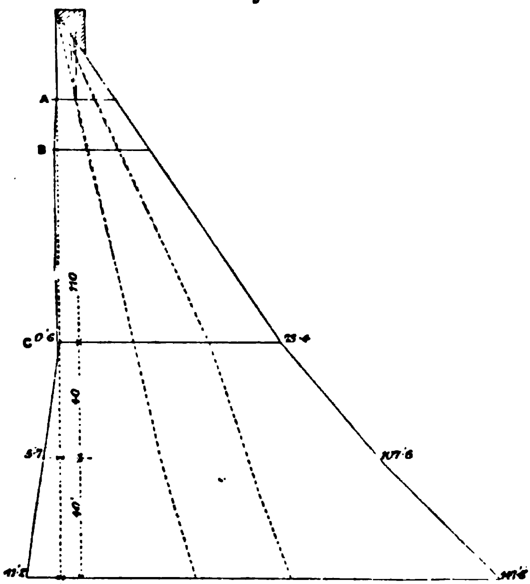
Mr. A. W. Brightmore. the conditions obtaining in notably successful dams. Bouvier had shown that the maximum stress in a dam was not on the horizontal plane, but on a plane perpendicular to the direction of the action of the resultant; and equalled the maximum vertical intensity of pressure divided by the cosine squared of the angle which the resultant made with the vertical. Rankine's and Bouvier's expressions being entirely empirical, caution was necessary in applying them to examples in which the elements of the problem, *i.e.*, the specific gravity of the masonry and the limiting intensity of pressure, were different.

Delocre's method of dividing a dam into parts had been criticised by Professor Kreuter. In this he did not, in Mr. Brightmore's opinion, go far enough, as the difficulty of treating the problem had chiefly arisen from the method of dividing up the dam. A more scientific plan appeared to be to treat first that part in which the maximum intensity of pressure was not attained at either the inner or outer faces. That might be called the "low" dam. The second part was that in which the maximum intensity of pressure was attained at the outer, but not at the inner face. That might be conveniently styled the "high" dam. The third part was that in which the maximum intensity of pressure was attained at both the inner and outer faces. Part 1 would include Professor Kreuter's first three divisions, and parts 2 and 3 would represent his fourth, for he did not distinguish between the attainment of maximum pressure at the inner and outer toes. In Part 1 the resultant pressures, resultant reservoir full and empty, fell at the outer and inner thirds of the breadths respectively, and, as Rankine had shown, simply required to be a triangle whose inner face was vertical, and of which the cotangent of the vertical angle equalled the square root of the specific gravity of the masonry. *Fig. 10* showed the section calculated—assuming the specific gravity of the masonry equal to 2·25 and the limiting intensity of stress equal to 10 tons per square foot. The triangular hatched portion had to be added at the top to resist wave-action, and sometimes to carry a road-way. That did not affect the conditions of the problem down to the point A, at which the vertical, through the centre of gravity of the triangular lump, cut the inner third of the breadth; below that point the resultant weight of the dam and triangular piece fell in the inner third, and continued to approach the inner face down to the point B, where, the weight of the triangular piece becoming insignificant compared with that of the dam, it receded from the face. A batter must therefore be given to the inner

Mr. A. W. Brightmore.

face between A and B. Below B, to the base of the "low" dam at C, the inner face might be vertical. The effect of the batter was to bring the resultant, when the reservoir was full, well within the middle third. Here, in Professor Kreuter's design, a system of paring was adopted, which, whilst it only slightly diminished the section, caused an overhang at the inner face, which not only violated the condition of "no tension," but was, in Mr. Brightmore's opinion, inadmissible in practice. In Part 2, introduction of the condition that the pressure be a maximum at the outer toe necessitated the resultant, reservoir full, being inside

Fig. 10.



the middle third. The proportions of Part 2 of the dam could be easily calculated by causing the resultant of the weight of the dam and of the water over its inner face to fall at the inner third of the breadth. This condition had no prejudicial effect whilst it delayed the attainment of the maximum pressure at the inner toe, until a height was reached exceeding that to which any dam had yet been built. The formula—

$$b = \sqrt{\frac{w\,h^3}{s}\left(1 + \frac{w^2\,h^4}{4\,W^2}\right)},$$

where b was breadth at a depth h below the crest, W was the

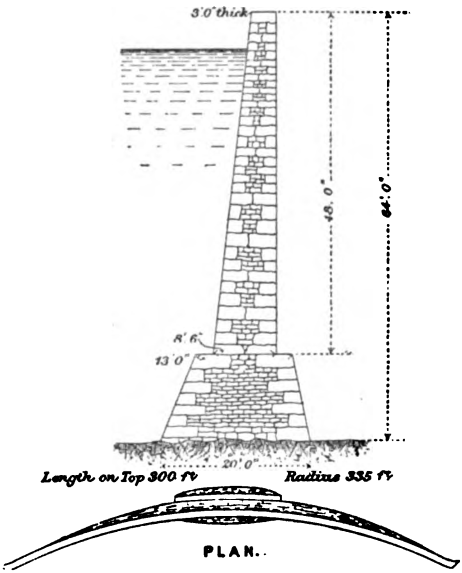
Mr. A. W. Brightmore. weight of masonry and water above b, and w was the weight of unit volume of water—was then easily rigorously deduced. It gave the breadth at any depth in terms of the depth and of W. At first sight it might appear that the presence of the latter factor would render the formula useless; but examination showed that the term containing W was only one-seventh of the whole expression; in fact, it ranged from one-sixth to one-eighth between the depths of 110 and 190 feet, so that if W was known even approximately, the breadth could be readily and accurately found. Thus, by considering successive laminæ, the breadths could be simply calculated, and by taking moments about the inner third of the breadth, the projection at the inner face was directly deduced without having to find either the centre of gravity of the dam or of the water over the inner face. That formula was readily adapted to very high dams—*i.e.*, to Part 3 where the limiting stress was attained in both faces; but as such dams were not under discussion, he did not propose to consider that now. In considering the above Part 2, Professor Kreuter had first found an equation to a profile—neglecting the weight of water over the inner face—with the maximum pressure at one face and the pressure zero at the other face. The curve thus found in no way satisfied the conditions of the problem, for, instead of having the resultant pressure at one-third of the base and the minimum pressure zero, it was obviously desirable to have the resultant pressure within the middle third and the minimum pressure as high as possible. By extending the triangular profile of the low dam downwards, and by an elaborate and ingenious system of projections, Professor Kreuter had deduced a breadth giving the pressures desired. But in Mr. Brightmore's opinion, that did not advance the investigation of the "high" dam at all.

Mr. W. Shelford. Mr. W. Shelford said in this country the storage of water was almost entirely confined to domestic supply, but in some foreign countries where the rainfall was less abundant and regular irrigation became exceedingly important, the quantities of water dealt with were much larger. For domestic supply 25 gallons per head per day might be considered to be the standard, or 9,125 gallons per head per annum. The same individual, if living upon irrigated land, would require at least 9 inches depth of water over an area approaching 2 acres, which would be 408,000 gallons per annum, or forty-five times as much as the former quantity. Any one who had been in Spain would know that the supply of water for the purpose of irrigation was largely conducive to the fertility of that country. In Egypt and in the East, irrigation was also extensively

employed. In other countries which were new and developing, and in which engineers were therefore more interested, there was the same want of water for irrigating; for example, in Australia, in the Western States of America, from Montana on the North to New Mexico in the South, in Mexico, and in the western provinces of the Argentine Republic, enormous tracts of arid land were found, which might be converted by a supply of water from a wilderness to a garden. With regard to the question of cost, the capital expenditure for domestic-supply works might be taken from £1 to £4 per head, *i.e.*, for 9,125 gallons per annum, and within those limits even Englishmen would not grumble if they got the water. It was totally out of the question to adopt such figures when considering the much larger quantity per head required for irrigation. The supply of irrigation water was not a sanitary but a commercial question, for a district which obtained cheap irrigation could send its produce to market successfully, whilst a district with dear irrigation was excluded. Consequently, it was very important to consider the cheapest mode of constructing storage reservoirs. In California, which might be taken as a type of the countries of which he was speaking, in which large tracts of land were now being brought into cultivation by irrigation-works, very bold attempts had been made to economise the storage of water; gravity dams had been in some cases discarded, and arched dams, *i.e.*, horizontal arches, adopted. The most remarkable was the Bear Valley dam, the section of which, *Fig. 11*, looked almost like an obelisk, 48 feet in height, with the top 3 feet thick, and the bottom of the obelisk only 8½ feet across. The length of the dam was 300 feet, its radius was 335 feet, and the total height was 64 feet. It stood on a granite bed, and abutted against the

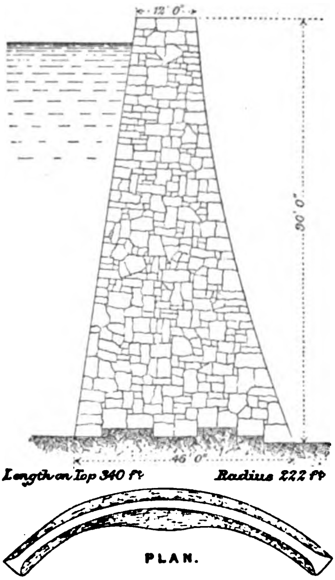
Mr. W. Shelford.

Fig. 11.



Mr. W. Shelford. granite sides of a steep ravine. The abutting ends of the dam were made twice the thickness of the centre. It was built in 1883, and had for some time been charged with water up to within 5 feet of the top, and had stood. During the first year it was noted that there was considerable efflorescence or sweating of lime through it. That, he understood, had disappeared to a large extent. The joints of the coping had opened slightly, and the movement had been attributed to the expansion and subsequent contraction of the masonry, which was rough ashlar granite on both faces, and coursed rubble hearting, all in cement. There was a leak also at the bottom of one end, which was said to have come through a rift in the granite rock. The dam had been so successful that a proposition had been made to extend the reservoir, and to increase the depth from 60 feet to 100 feet, in which case, if successful, it would be an exceedingly valuable property. Other dams shown in the diagrams, *e.g.*, the Sweet Water, 90 feet high, *Fig. 12*, were not gravity dams, but were arches with abutments. They were less bold than the Bear Valley dam, which, he believed, was the most remarkable in the world. It was right, however, to point out that a heavy responsibility lay upon the designer of an arched dam in the present state of knowledge; for, in order to close a valley, it might be necessary to make an arch of much larger dimensions than any vertical arch yet constructed of masonry, and the pressures and movements due to variation of temperature would be greater than in existing vertical arches. The stresses would also differ from those in gravity dams by reason of the abutments being unyielding. Arched dams would also tend to move under the changing pressures due to the drawing out and refilling of the reservoir, and might thus become weakened, or even ruptured, in time. The standard authorities appeared to agree that in long and high dams the gravity dam was alone applicable. The Bear Valley,

Fig. 12.



the Sweet Water, and the Zola dams, with respective lengths of 300 feet, 340 feet, and 205 feet, were not of any considerable length; but the Sweet Water dam, 90 feet, and the Zola dam, reported to be 126 feet in height, were comparatively high. The great value of the Bear Valley dam was shown by a comparison with others. For instance, the Quaker Bridge dam at New York, which was the highest built, or building, being 270 feet, was a gravity dam designed after a great deal of careful study both by engineers and by a commission of experts, and might be taken as a type of the gravity dam. The storage was 31,250,000,000 imperial gallons. The estimated cost per million gallons was £26 8*s*. 10*d*. The Sweet Water dam contained 4,902,000,000 gallons, at a cost of £9 17*s*. 4*d*. per million gallons. The Bear Valley dam contained 10,990,000,000, at a cost of £1 8*s*. 3*d*. per million gallons. Comparing with that the Jeypore sandbank, which was nothing if not a cheap dam, the contents were 925,000,000 gallons, and the cost was £7 15*s*. 0*d*. per million gallons. He only wished to add that if an efficient dam could be made between such wide limits as £1 8*s*. 3*d*. per million gallons and £26 8*s*. 10*d*. per million gallons stored, there must still be ample room for engineers to display their skill in the design and construction of storage reservoirs. Mr. W. Shelford.

Mr. ALEX. R. BINNIE could not but allude to the success which had attended grouping together similar Papers on cognate subjects. Here were three storage reservoirs, one with a masonry, one with an earthen, and one with a sand embankment; and also a Paper on the theoretical consideration of the design of masonry dams. Turning to other matters relating to the storage of water, such as the flow off the ground, the floods and the evaporation, great diversity was found. With regard to the flow off the ground, the mean rainfall was about 102 inches at Tansa. It had been assumed that one-third would come off. He was inclined to think that this was a low estimate, for the reason that he had gauged from a similar trap and basalt area in India, considerably smaller certainly, a flow of 40 per cent. due to a rainfall of 39½ inches. That was not, however, the way to look at flows from the ground; the question must be regarded a little more in detail. Of course it was made clear in the Paper that the drainage-areas at the commencement of the rainy season were perfectly dry, so that they started every year on a common basis. In the case of Nagpur there was a rainfall of 6·77 inches in June, of which only 4·7 per cent. flowed off the ground. In July, with a rainfall of 12·7 inches, 22·7 per cent. flowed off. In Mr. Alex. R. Binnie.

Mr. Alex. R. Binnie. August, with a rainfall of 11·82 inches, 55·8 per cent. flowed off, and in September, with a rainfall of 7·99 inches, 74·4 per cent. flowed off. Therefore, if the rain had increased from that 39 inches and upwards, a point would soon have been arrived at where the drainage area would become almost saturated. He had seen the Tansa basin, though he had not examined the whole of it; and it was of a character very similar to that of the hills above Nagpur. It might be much more humid, and in that case a much heavier discharge would be expected. Turning to the subject of floods, Mr. Mansergh had stated that Mr. Binnie had measured some floods. The Tansa flood was 744 cubic feet per second per 1,000 acres. He had measured one flood that ran up to 770 cubic feet per second per 1,000 acres. He wished to draw attention to what had happened on that occasion, which was due to a rainfall of 2·2 inches in one hour and twenty minutes. He had also measured a shower of 3·55 inches which fell in forty-five minutes. That was at the rate of 4·73 inches an hour, and yet only gave a discharge of 687 cubic feet per minute. What was the reason that the smaller rainfall in a longer time produced the larger flood? It was simply that the ground was saturated by previous rain, whereas before the heavy fall there was an intermediate period of dry weather. That matter was exceedingly interesting, because coming to another important factor, the saturation of ground, it was necessary to inquire what was the greatest rainfall in any twenty-four hours. With a mean rainfall of 40 inches at Nagpur, the heaviest recorded rainfall in twenty-four hours was 18 inches. Doubtless, there had been similar heavy rainfalls in the neighbourhood of Bombay. With regard to the Table giving the results of gauging the Surya River, there did not appear to be either rhyme or reason in the proportion flowing from the ground when compared with the total rainfall. The Table would be interesting and lucid, if it were treated month by month, as had been done at Nagpur. If, instead of saying in 1889 there was a rainfall of 45·79 inches and only 13 per cent. flowed off the ground, the details had been given, it would probably be found that the 45 inches fell during the monsoon months, and in sudden and heavy showers; that those showers falling on dry ground, were largely absorbed, and afterwards, probably, evaporated; that then the ground got dried again, and so on in succession. Turning to Jeypore, where only one-sixtieth of the total flow was discharged, that did appear somewhat surprising, but not extremely so, seeing that the Author had stated that the gauging of smaller streams in the district only

gave one-tenth. How was this to be accounted for? Very simply, by the fact that the district was sandy. Those who gave attention to questions of the water flowing off the ground must gradually come to the conclusion that a porous and absorbent soil was one of the soils yielding the smallest amount of water; and for this reason, that when heavy rain fell on a sandy soil it was quickly absorbed and held in the upper part of the sand by capillary attraction. When the sun shone upon it during the summer months, either in this country or in the tropics, a large portion was evaporated, and consequently a very small portion of the rainfall flowed off the ground. Quite apart from the loss due to evaporation from the surface of the ground, which effected the available flow from it, in India another factor, viz., the evaporation from the free-water surface of the reservoir during the long dry period of eight or nine months, required to be taken into account. In the cases of Tansa and of Baroda, 6 feet had been allowed for under this head, which he thought a safe amount. The Author had given the evaporation at Tansa as 2·94 feet to 2·96 feet during the 151 days, January to May inclusive. In the much drier climate of Nagpur, from very careful gaugings made by him, the evaporation from the 10th October, 1872, to the 9th June, 1873, 242 days, amounted to 4 feet. How important a question evaporation from the water-surface was in India might be judged from the fact that at Nagpur it exceeded 50 per cent. of the total fall of storage during the dry season 1872–73. In the case of Tansa, 6 feet evaporated would approach one-third of the total quantity collected. As previously pointed out by him, evaporation depended more on the comparative humidity of the air than on the actual temperature. He had intended to offer some further remarks on the construction of the works referred to in the three valuable Papers, but time did not permit him to do so. With regard to the sand embankment, the use of the word "sand" scarcely conveyed the meaning of the Author. What he had said was that there was sand and mud. If the bank was made of pure sand it would act like a filter and the water would come through. There must be a considerablo amount of silt combined with the sand which formed a cementitious material, preventing rapid percolation. Probably there was considerable percolation, but as the stream feeding that reservoir carried silt, it might be hoped that the upper part of the sand would become perfectly watertight. Mr. Alex. R. Binnie.

Mr. CHARLES HAWKSLEY pointed out that a most important factor in the design of masonry dams and similar structures was Mr. Charles Hawksley.

Mr. Charles Hawksley. the factor of safety to be adopted. Great diversity of opinion existed among engineers on that subject, but his late father had always held the opinion, and had acted upon it in his designs, that a very ample margin ought to be allowed to cover those differences in the strength of the materials and defects in workmanship which always occurred, and especially so in cases in which the life and property of others was involved. When making the design for the Vyrnwy dam, which had been referred to in one of the Papers, his father had aimed at securing a factor of safety of 2, and he believed that had been closely approximated to. There was another point to which great attention had been given, and which had not been referred to in any of the Papers, viz., the question of up-lifting action of the water on the bottom of the dam, to which it might find access through fissures or in some other way. That was a question of great importance, the neglect of which he believed had been the cause of the failure of some of the masonry dams which had been destroyed. To show the importance of it, he might mention that in the case of the Vyrnwy dam, the overturning pressure due to up-lift was greater than that due to the pressure of the water on the face of the wall. Another point which had not been referred to in the Papers was the question of the submersion of part of the structure when, as in the case of the Vyrnwy dam, the water was upheld on the lower side of the dam to the rounded nose at the bottom of the dam. All the masonry below that was immersed in water, and therefore, of course, its weight was virtually greatly reduced. He ventured to think that those were two very important points which should not be lost sight of in investigating structures of that class.

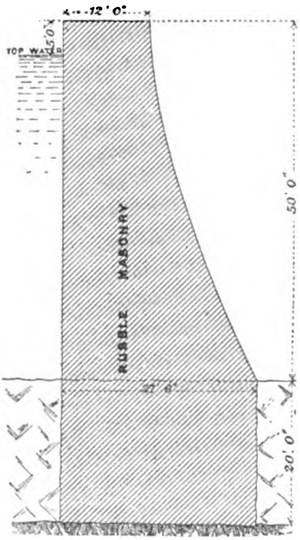
Mr. R. Hassard. Mr. R. Hassard said that the velocity and volume with which water during heavy floods would flow off steep mountain slopes into the reservoirs supplying the large towns in this country was a matter of great moment. He had seen two or three very destructive floods which, although not so great as those which had occurred in tropical countries, at all events approximated to them. On the 13th of October, 1891, in a valley near Dublin, where he and his partner about two years before had constructed some waterworks, a most violent flood had occurred. The area draining to the reservoirs was 6,940 acres; the valley trended from south-east to north-west, and was the first deep valley on the western side of the mountain district which lay immediately to the south of Dublin, being at right-angles to the prevalent winds. The longitudinal inclination of the valley where the reservoirs were

constructed was about 1 in 60, so that an embankment 66 feet in height would make a reservoir about one mile in length. On the western side of the valley the hills were steep, the average slope from the summits to the reservoirs being 1 in 4 or 1 in 5. At the upper end of the valley the hills attained altitudes of 2,364 feet to 2,475 feet, giving an average inclination of 1 in 8 to 1 in 9, and on the east side of the valley the slopes were also steep, averaging about 1 in 9. For some days previous to the occurrence of that flood, the weather had been wet, the ground was saturated, so far as was possible in such steep ground, and the reservoirs were both full, so that there were present all the circumstances and conditions which tended to produce sudden and severe floods. The rainfall which caused this particular flood commenced at seven o'clock in the morning, and continued steadily until three o'clock in the afternoon, when it ceased rather suddenly; the flood attained its maximum between one and two o'clock in the day, and at that time, over the waste weir of the lower reservoir, which was 200 feet in length, the water was pouring to a depth of 2 feet 10 inches; and, in addition, there was the discharge through two outlet-pipes, respectively 27 inches and 24 inches in diameter, fully open and under a head of 53 feet—giving a total quantity of 214,000 cubic feet discharged per minute. That was equal to 514 cubic feet per second per 1,000 acres flowing off the ground. The rainfall, as registered by a gauge placed between the two reservoirs at an altitude of 512 feet in the lowest part of the drainage area, was 1·60 inch during the eight hours. But, as the drainage-area lay considerably above that point, he believed that it did not indicate nearly the true amount of the rainfall; and that it must have averaged at least 2 inches over the whole of the drainage-area during the period of the storm. The 2 feet 10 inches in depth measured on the side wall of the waste weir did not, of course, indicate the true head of the water discharged over it; and he had estimated that that would give a discharge of something over 533 cubic feet per second per 1,000 acres. Mr. R. Hassard.

He had been at some trouble to investigate the comparative cost of earthen embankments and of masonry dams for impounding water. Taking the statements in the first Paper as correct, and the value of the rupee at two shillings, he made the cost of the masonry in the Tansa Dam 14*s.* 8*d.* per cubic yard; the cost of the Furens dam, built some forty years ago, had been given as 14*s.* per cubic yard; but such work could not now be executed at those prices in this country. He considered the price of rubble

Mr. R. Hassard. masonry for such works set in hydraulic mortar or in Portland cement mortar would be 25*s.* per cubic yard. For a site such as that of the Tansa dam, where it was considered necessary to excavate foundations, and to re-fill the excavation with masonry, *Fig. 13*, it was obvious that in the case of an embankment, all that would have been necessary was to strip the surface and sink a trench under the centre of the embankment, *Fig. 14*, to be re-filled with concrete or puddle, or with both, to the subjacent amygdaloid trap rock. The masonry buried in the foundations of the Tansa dam was three-fifths of the total quantity in the structure, and must have formed an important item of the cost. On such a site as the above, an embankment 50 feet above the surface of the ground would, at the prices ordinarily paid in this country, cost about £113 per lineal yard against £204 for a masonry dam. Similarly an embankment 75 feet high, would cost £202 per lineal yard against £381 for a masonry dam. No doubt there were many cases where embankments would not be safe or practicable, but up to 70, 80 or 90 feet in height there was not any difficulty, where the material was suitable, in constructing safe and water-tight embankments.

Fig. 13

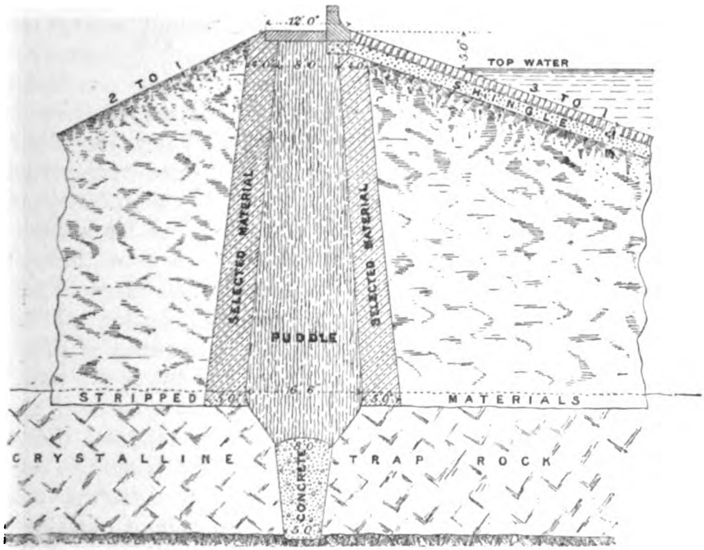


The outlet-culverts should always be placed on solid ground at the side of the valley and at such a depth as to pass completely under the concrete or puddle trench forming the water-tight barrier so as to leave it intact. Difference of opinion existed as to whether it was preferable to construct an outlet-culvert by tunnelling or in a deep trench. He much preferred the latter course, so that everything connected with the outlet might be constructed in the open day, and under better supervision than was practicable in tunnelling. Almost all the failures of embankments with which he was acquainted had arisen from defects in the outlet-culverts, either in design or workmanship, and not from instability or weakness in the main structures.

Mr. Clerke. Mr. CLERKE, in reply to Mr. Russell Aitkin's reference to the Shewla project, did not think any advantage could accrue

from discussing the relative merits of the Tansa and the Shewla projects. The former was an accomplished work, whereas the other existed only on paper. In answer to Mr. Mansergh: The Tansa Valley was bounded on the north and south by ranges of hills rising from 800 to 1,700 feet above the surface of the lake. At its eastern end, these ranges of hills died out, and the boundary of the catchment-area, for a length of about 3 miles, could not be fixed by mere inspection. The slopes of the hills were well wooded. The nature of the ground was hard, being muram (disintegrated trap rock), except for a narrow strip in the upper Mr. Clerke.

Fig. 14.



part of the valley where the ground was capable of cultivation. There were numerous ravines on the slopes of the hills, which brought the rainfall rapidly down to the basin. The dam was completed in April, 1891, and on the 25th June the water in the lake stood at R.L. 375·50. The lake-surface then rose slowly until the 18th July when it stood at R.L. 381. It then rose rapidly, and on the 30th July it began to spill over the waste-weir at R.L. 405. On the 4th August the lake-surface reached R.L. 406·95 or 1·95 foot above the crest of the waste-weir. This was the maximum flood which he had observed, and amounted to

Mr. Clerke. 17,000 cubic feet per second. The rainfall, as gauged at the side of the dam at that period, was:—

	Inches.
31st July	0·91
1st Aug.	1·86
2nd „	0·61
3rd „	4·52
4th „	2·14

The incidence of floods showed that the rainfall at the dam could not be taken as an index of the general rainfall on the catchment-area. In the early part of September, 1892, the rainfall gauged at the dam site was:—

	Inches.
1st Sept.	5·31
2nd „	3·19
3rd „	3·96
4th „	3·78

On the 4th the lake-surface was only 1 foot above the crest of the waste-weir; that was the maximum flood of that year. The only difference between the cross-section of the waste-weir portion of the dam and the general cross-section, as shown on the diagram, was that along the waste-weir portion, 1,650 feet long, there was on the inner edge a fine ashlar coping, dressed to a quadrant of 6 inches radius. The heaviest flood which had passed over the waste-weir was that of the 4th August, 1891, during which the water in the lake rose to 1·95 foot above the crest of the weir. That flood did not fall directly on the ground below the weir, but took the curved face of the wall in its fall. At a depth of 20 feet from the top the curved face projected 7 feet beyond the outer edge of the waste-weir. There was not much erosion of the ground immediately below the weir. The greatest erosion occurred some distance from the face of the weir, where the flood became concentrated in the narrow passages with steep falls, by which it found its way back to the old river course. The foundations of the waste-weir were in solid rock at depths varying between 8 feet and 40 feet below the surface of the ground. In his opinion it would be some time before any protective works were necessary at the toe of the waste-weir, though in the course of time, as the channel by which the water passed away eroded retrogressively towards the face of the weir, some works might be required. The two lines shown on the longitudinal section on the axis of the dam below the ground line were explained on the Plate. The faces of the rubble side-walls in the cut-and-cover were not very rough. Most of the stone used had a fairly smooth

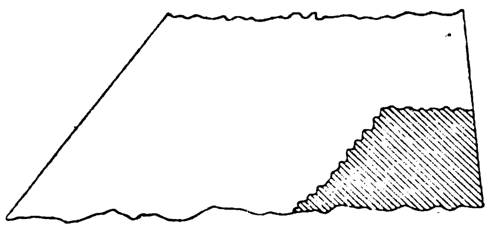
fracture, and the projections and depressions from the inner surface of the wall would average about 1 inch. As suggested by Mr. Mansergh, he had exhibited two books of photographs taken when the works were in progress, and had marked places where the faces of the side-walls in cut-and-cover are shown. When the conduit was tested up to the springing level, the side walls were found, on the whole, to be fairly water-tight. In places there was a little weeping, and in a few instances fine jets of water issued, indicating a want of careful workmanship in the building in these places. Mr. Clerke.

He had not wished to undervalue the late Professor Rankine's researches, and in answer to Professor Unwin desired to explain that he had known more than one instance where, for low dams under 90 feet in height, Rankine's general section had been adopted without any reduction and an unnecessary expenditure of material had been entailed. He had therefore thought it desirable, for the sake of younger members of the profession, that this point should be brought prominently forward. With reference to Professor Unwin's remarks on the experience recorded with cast-iron mains laid above ground, he would explain that the pipes were laid with a clearance of one-eighth inch; and, as a fact, every joint was an expansion joint—that they so acted could be seen by a close inspection of the spigot of each pipe under variations of temperature. The danger which was apprehended in some quarters, as pointed out in the Paper, was that on gradients the tendency of the alternations of expansion and contraction would be to cause the line of pipe to creep down the gradient.

With reference to Mr. Symons's remarks on questions of loss and questions of evaporation. Observations for loss by evaporation alone had been made some years ago on storage reservoirs in the Deccan, India, under the direction of General Fife, R.E., and more recently on reservoirs of the Bombay Water-works, which were at present under the charge of Mr. S. Tomlinson, M. Inst. C.E. The figure of 72 inches made use of in his Paper and in that on the Baroda Water-works, was based on the results of the observations made in the Deccan. The share of the loss due to leakage would vary widely with the circumstances of each particular case. He regarded leakage, which included percolation through the work and through the underlying strata, as the only other source of loss, for when once the bed of the lake had become fully saturated, no further loss from absorption could accrue. In the case of the Tansa lake, the leakage was so small that he considered it might be disregarded, taking into account the

Mr. Clerke. large surface of the lake. He therefore thought that the figures given in the Paper might be taken as the measure of evaporation alone for the periods to which they referred. In the latter part of his remarks, Mr. Symons appeared to have applied the annual evaporation to the rain falling on the whole catchment-area of 52·5 square miles. That did not appear to be correct. Mr. Clerke had assumed that two-thirds of the rainfall was lost. That loss was due to absorption and evaporation from water lying on the surface of the ground before it found its way into the lake. He had assumed that one-third of the rainfall would be collected in the lake, and after being so collected, he had allowed for the annual evaporation on the mean area of the lake-surface. From the remarks made by more than one of the speakers, it would appear that the floods provided for were regarded with surprise amounting almost to incredulity. But engineers accustomed to deal with the conditions obtaining in India would not so regard them. He had been connected with the construction of numerous storage-works in that country, with catchment-areas varying between 196 square miles and 3 square miles. In the largest area provision for a flood equal to a discharge of ½ inch per hour had been made, and in some of the smaller ones for a discharge of more than 2 inches per hour. Experience gained during many years had led him to the conclusion that such provision was not excessive. He admitted that he should, for purposes of comparison, have raised the Tansa section 5 feet higher, so as to make its waste-weir level correspond with the top of the Vyrnwy section. From Mr. Deacon's remarks on the construction of the Tansa dam in the first season, he feared he had not clearly explained what was then executed. The shaded portion, *Fig. 15*, was built in the first season; in the second season the section was completed to the full width, and brought up to the height shown in the sketch. As regarded the circumstances under which the observations were made, which gave the discharge of the siphons at 21,500,000 gallons. When the works at the head of the conduit had been completed, a length of 2½ miles of the conduit was blocked up by a cross-wall. The capacity of that

Fig. 15.



portion was determined with accuracy. The blocked-up portion was filled from the head-sluices and several observations were made to determine the coefficient of the sluices. Those experiments gave the coefficient as 0·68. That coefficient might vary within small limits, according to the head of water over the sluice and the area of sluice-way open. The cross-wall was removed and water was admitted into the conduit from the head-sluices—the quantity admitted was calculated from the results of the observations mentioned. Numerous experiments were then made, by gaugings, to determine the coefficients applicable to the conduit and the result of those had been stated in the Paper. Observations were then made to determine the maximum conveying-capacity of the 48-inch siphons; these were made on siphon No. 6, which was 11½ miles in length. The water in the conduit at the north or upper end of the siphon was flowing at a uniform depth of 3·50 feet. In the conduit at the south, or lower end, the uniform depth was 3·60 feet; those figures showed that the siphon was conveying very nearly its maximum quantity. Applying Kutter's formula, with the coefficient 0·013, to the conduit with a depth of 3·60 feet flowing in it, the discharge worked out to 21,900,000 gallons. It had been observed by Mr. Binnie that the assumption that one-third of the rainfall would flow off the ground was low. That was true, but in dealing with questions of that nature, some assumption had to be made, and it was best to err on the safe side in making a forecast. The ratio of the discharge to the rainfall could not, even in the case of any one locality, be taken as definite. It was dependent on the character, as distinguished from the quantity, of the rainfall of the year under consideration. Mr. Clerke.

Colonel S. S. Jacob, in reply to the criticisms on the Jeypore Waterworks, was not surprised to learn that the dam had raised doubts in the minds of engineers. It had been long before he had ventured to adopt the construction of a sand dam, but experience gained elsewhere encouraged the attempt. The circumstances of the case were exceptional; the ground around was so porous that any attempt to make a water-tight dam would have been useless. Leakage was expected and measures were taken to provide against it. The object in view was to store a portion of the water which, in the rainy season, went to waste, and by storing it in the only place available, viz., the sandy bed of a sandy nullah, to form a reservoir which would furnish a supply until, at least, the following rains, leakage from which would prevent the springs in the nullah below it from drying up in the hot season, and so afford a constant supply to the pumping-station, situated a short distance Colonel Jacob.

Colonel Jacob. lower down. No other kind of dam was so suitable in such a place or could have been made at so small a cost. Any one not conversant with the local conditions, or who had not seen works of the nature described, would no doubt be disposed to condemn the idea; but those who had seen the work and who know how perfectly it fulfilled its purpose, had acknowledged that, if it was bold, it was a thoroughly successful work. Further information was desired on the question of how such a small discharge could supply the water, which, it was stated, was required for consumption. It was true that calculation, after a heavy fall of rain, showed that at the time only about one-sixtieth of the rainfall was discharged, because the soil of the drainage-area was so very sandy; but as the bed of the nullah was about 60 feet below the natural surface of the ground, it acted as an arterial drainage-line and was fed for weeks, after the rain had ceased, by springs which derived their water from rain which had been absorbed as it fell, and subsequently gradually percolated into the bed of the nullah.

Prof. Kreuter. Prof. Kreuter, in reply to criticisms of the principles upon which he had founded his theory, and upon which indeed all theory of structures had to be founded, referred to the valuable information given by Prof. Unwin and to the lucid explanations of Mr. A. W. Brightmore. The title of the Paper had been said to be rather comprehensive, and it was possible that, as a foreigner, he had not chosen the best expression for it. His intention was to urge that the design of a structure consisted chiefly in determining its correct form and dimensions. That problem, in his opinion, could never be solved empirically in a satisfactory manner, but only by the aid of scientific investigations, in the case of important structures, the cost of which was enormous and the failure of which might lead to terrible disasters. Scientific treatment was indispensable when the action of forces acting upon such structures was not of a very simple description. It was obvious however, that the man to be trusted with the design of a masonry dam and its appendages must not be a pure mathematician, but he must combine scientific knowledge and practical experience.

He knew Mr. Bouvier's Paper, but could not find any connection between it and his own. That gentleman had himself said, in the third chapter, that he only intended to complete and to verify some of Messrs. de Sazilly's and Delocre's deductions. For the rest, Mr. Bouvier had started from a profile previously assumed, and thus did not seem to have advanced the solution of the problem. The French engineers had not fixed the condition

that there should be no tension anywhere at the faces of a dam. They had only required that the intensity of pressure at the faces should at no point exceed certain limits. That condition has led to a theoretic type in which both lines of pressure fell within the middle third, in the case of a maximum working pressure of 6 kilog. per sq. cm., in the Furens dam. In the theoretic type for the dam of the Ban, however, which had been calculated by Mr. Montgolfier, and where 14 kilograms per square centimetre had been assumed, the consequence of the above condition was a very considerable deviation of the line of pressure, reservoir full, from the middle third. That uncertainty was the reason why Rankine found it advisable to premise the necessity that the lines of pressure should not leave the middle third of the profile. Prof. Kreuter.

As to Mr. A. W. Brightmore's objection, that, by making a part of the inner face overhang near the top, the condition of no tension was violated, he would reply, that that tension was practically nil, the corresponding deviation of the line of pressure. It was therefore not to be compared with the deviation of the line of resistance, reservoir empty, in the Tansa dam. He was aware of the circumstances which still ought to have been noticed in order to arrive at a complete solution of the problem, but was sure that it would be of no further practical advantage to enter into too minute details; and, on the other hand, he thought that any expansion of the theory might be arrived at without difficulty.

Correspondence.

Mr. Marius Bouvier considered that some interest might attach to French practice in regard to such structures as those alluded to in the Papers. This was exemplified by the reservoirs in the south of France, an account of which had been recently given by him to the *V^me^ Congrès Internationale de Navigation Intérieure*,[1] held in Paris in 1892. Mr. Marius Bouvier.

Dr. A. von Braunmühl, of Munich, observed that, hitherto, in selecting the most suitable form of masonry dam, either an empirical method had been adopted or a more or less arbitrary section had been accepted; but Professor Kreuter had presented a complete, consistent, mathematical treatment of the subject. An Dr. A. von Braunmühl.

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[1] "Reservoirs in the South of France." By Marius Bouvier. Fifth International Congress on Inland Navigation, Paris, 1892.

Dr. A. von Braunmühl.

analysis of the problem had convinced Delocre that, in consequence of the varying relation of the pressures, continuous profiles, such, for instance, as logarithmic curves, were not possible; he therefore divided the cross-section into three portions. Professor Kreuter remarked truly that, in order for the line of pressure to change from its fixed position in portion No. 1 to its position in portion No. 2 of Delocre's section, an intermediate part should be introduced. This part was of trapezoidal form. The equation for the breadth in the third portion was given as an algebraic expression, that in the fourth (base) portion being determined by a logarithmic curve. The principal difficulty consisted, after the breadth in the entire cross-section of the dam were found, in so joining together the four portions as to ensure the fulfilment of the necessary conditions of equilibrium. This was an easy matter in the first and second portions, but in the last two the equation contained an integral that could only be evaluated step by step. The effect of the vertical component of the water-pressure on the profiles, hitherto neglected, was next considered by a combined graphical and analytical method.

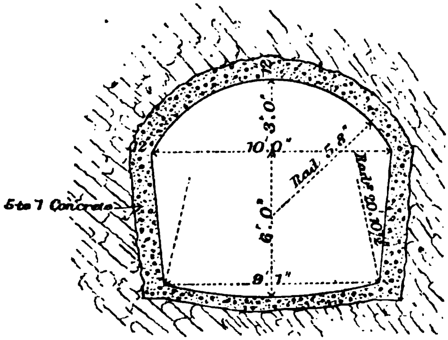
Engineers might, perhaps, consider this theoretical treatment of the question too cumbrous for ordinary use. Two replies might be given to this: first, the work involved in the computation was less laborious than it might at first sight appear to be; and, secondly, the time spent in arriving at a solution of the question was of little importance in relation to the certainty introduced into the design of dams by rigorous mathematical treatment.

Mr. A. Fairlie Bruce.

Mr. A. Fairlie Bruce observed that the strength of the Tansa dam might have been increased had it been constructed with an inner skin of 5 to 1 Portland cement concrete, say 5 feet thick at the bottom, diminishing to 1 foot at the top. That would not have added much to the cost, and would have prevented water from finding its way into the heart of the structure, which, while it might cause little or no leakage, must, by its pressure, impair the stability of the dam. It would have been interesting to know why rubble masonry was preferred to rubble concrete for the work. The crushing strength of the mortar compared unfavourably with that of Portland cement mortar. With cement concrete two weeks old, in which the mortar was composed of 1 part of cement to 3 parts of sand, he had, in arched ribs, obtained nearly as good results as those quoted in the Paper at six to eleven months; and at twelve weeks he had obtained 2,447 lbs. per square inch, which exceeded the maximum Tansa test. Judging from the quantity of water delivered, and the small loss by leakage, the workman-

ship of the aqueduct must have been of high class; but the section adopted seemed to leave something to be desired. He preferred that adopted on the new aqueduct from Loch Katrine, *Fig. 16*, in which the side walls had a batter of 1 in 12, and the invert had a versed sine of 6 inches. The flat invert of the Tansa aqueduct would be found difficult to cleanse, and silt would accumulate in the unlined portions. In the Loch Katrine aqueduct, a concrete invert was provided throughout the entire length, whether lined or not. As regarded the discharge of the siphon pipes, Mr. Bruce had found by experience that D'Arcy's formula gave a correct result, if a deduction of 4 per cent. were made for the resistance of curves. Applied to the Tansa case, it gave a delivery of 22,488,000 gallons, which corresponded closely with the result obtained by the Author. The subsequent diminution of delivery due to corrosion, unless the water was very soft, amounted to about 1 per cent. per annum. Considerable future trouble and expense might have been saved had a few lengths of the second line of pipes been laid at each of the valve-chambers, and closed with a blind flange, the sluices being afterwards fixed when required. The faucet adopted might have been made lighter had a steel hoop been shrunk on it. The thickness of lead (½ inch) was greater than that generally used. The usual practice now was to dispense with the bead on the spigot end, a steel or wrought-iron hoop being shrunk on instead, giving greater strength for transit, and capable of being afterwards struck off and sold for about its original cost. Had this been done, a shallower faucet might have been adopted. On the Glasgow Waterworks cant-collars were employed in place of bends on large pipes, and the convex sides of all curves were buttressed with masonry or concrete. For foreign work, where, as in the case of the Tansa works, transport presented a serious difficulty, steel offered many advantages over cast-iron for pipes. Referring to the Baroda Waterworks, the Author had made a great mistake in dispensing with a puddle-wall in the embank- Mr. A. Fairlie Bruce.

Fig. 16.



LOCH KATRINE AQUEDUCT.

Mr. A. Fairlie Bruce. ment, however good the material might have been. It would have been far better to have dispensed with the second puddle trench. The puddle in the trenches did not appear to have been worked with sufficient care. With layers of 6 inches to 12 inches thick, tramping alone was scarcely enough without cutting and working with proper tools, to amalgamate each layer with that below it. Under the circumstances, it was not surprising that considerable leakage had resulted, and it might tend to increase, till it eventually destroyed the embankment. *Fig. 6*, page 47, did not indicate whether the outlet-culvert was constructed in a trench or merely on the surface of the ground; nor how it was carried across the puddle-trench. If that was imperfectly done, it would introduce an additional source of danger. It was not evident why scrap-iron filters were considered necessary. Surely sand-filters alone would have been sufficient. If, in these latter, 12 inches of gravel, graduated from the size of shot downwards to that of marbles, had been substituted for 2-inch metal, it would have been better. As it was, the sand would be found to wash down through the stones, and they would become choked, and do harm rather than good. It was not made clear how the rate of filtration at the Jeypore Waterworks was regulated; nor was the capacity of the clear-water tank given. The latter should be such that, when the filter became fouled, there should always be sufficient storage for the filtered water when the engines were not running; and it should not be possible, by the pumps supplying more water than the filters could pass, to cause an excessive head on the latter, otherwise the sand would certainly be liable to break up. Mr. Bruce thought, for waterworks purposes, Worthington engines produced less shock in the pipes, and were more economical in working, than beam-engines.

Mr. Collignon. Mr. E. Collignon, of Paris, in reference to Prof. Kreuter's Paper, observed that much importance attached to the condition that the line of resultant pressures should always lie in the middle third of the section. Although it was desirable that the then resultant pressure, reservoir full, should not fall too far from the inner face, it was less important to prevent opening of the joints at the outer face, because the water could not take advantage of it there. On that account no inconvenience arose from causing the curve of pressure to approach the inner face slightly beyond the position assigned to it by the Author. The Author had confined himself to the consideration of horizontal joints, and had not dealt with imaginary joints situated obliquely in the masonry. In that respect, the necessity for considering oblique joints was admitted

by the majority of French engineers. Occasionally the limit of resistance, whilst low enough on a horizontal joint, attained a much higher value for an inclined joint. It would be well to introduce this correction for the special section proposed by the Author. Mr. Collignon.

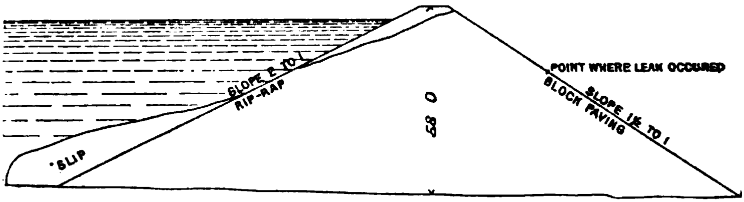
Mr. J. J. R. CROES considered that the methods employed in the construction of the Indian dams were interesting, but were certainly not applicable to any part of Europe or America; and the results in securing water-tight structures did not appear to have been such as would be satisfactory in American practice. Sufficient time had not elapsed to enable a judgment to be formed as to the stability of the Tansa dam, or as to its resistance to the percolation of water. The Author's contention that it was not essential that the mortar used in the interior of a large masonry-dam should be of a strength and hydraulic character equal to the best Portland cement, appeared to be reasonable. The crushing-resistance of mortar used in a dam could not be as great as that of the stone used in the masonry, nor could its specific gravity be as high. In the heart of a large mass of masonry, weight and homogeneity of the mass were the principal objects to be secured, and those could be best obtained by the use of as little mortar as was required to fill all the void space between the stones by a material which would resist solution by water. The practical limit to the use of large stones was governed by the possibility of roughly filling all the void spaces between the stones, and the best masonry-wall was one in which the stones were of large mass, imbedded in good hydraulic mortar, and laid sufficiently far apart to admit of the joints being filled with concrete composed of broken stone and good hydraulic cement, such as the Indian kunkur and the natural hydraulic cements of America. With rock weighing 172 lbs. per cubic foot, 32 per cent. of the mass might be of large stones and the specific gravity of the masonry would be 2·33. The earthen-dams at Baroda and Jeypore did not appear to have been constructed with a view to impermeability, which was essential in reservoirs in which it was important to retain all the water impounded. The cross-section of both of the embankments showed, moreover, that economy of space was not an object in their construction. The slope on the water-face, of 3 to 1 in one case and 4 to 1 on the other, was greater than that ordinarily permissible in reservoir embankments. Mr. Croes.

Such flat slopes possessed advantages in the case of sudden emptying of a reservoir, when a tendency occurred for the inner face of the embankment to slide, owing to the water retained within it. Two instances of such sliding of the inner bank of reservoirs

Mr. Croes. had come under his notice during the past year. At Pottsville, Pennsylvania, a storage-reservoir was constructed in 1886. The embankment was formed of earth, 500 feet long on the top and 60 feet above the natural surface of the ground at the middle. It was 10 feet wide; the outer slope was 1½ to 1, paved with stone 8 inches to 12 inches thick; and the inner slope was 2 to 1, covered with rip-rap.

The reservoir was filled but not drawn down for six years. In August, 1892, a leak appeared at the middle of the embankment on the outer face, about 20 feet below the top. The water was turbid, carrying with it fine sand. The leakage increased rapidly until it was said to have flowed down the slope in a stream 3 inches deep and 20 to 30 feet wide. The waste-valves were opened and the water was drawn off the reservoir as rapidly as possible. Two days were occupied in emptying the reservoir. When it was nearly empty, the inner slope settled and slid down

Fig. 17.



into the basin, causing a depression in the dam about 5 feet deep in the middle and 200 feet long. At the foot of the bank the material slid out about 40 feet, assuming the form shown in *Fig. 17.* This embankment was stated to have been constructed of selected material excavated from the hill-sides in the vicinity, consisting of clay, sand, and a small quantity of gravel—the clay predominating. The material had been hauled on to the embankment and spread in thin layers and consolidated by the passing of the teams and wagons, but was not rammed or rolled. Through the middle of the embankment specially selected material was placed which was wet and cut with spades. Examination of this bank a year after the break occurred showed it to be charged with water. It had taken six years, after the reservoir was filled, for the entire dam to become saturated with water below a plane sloping from the water-surface inside to a point about 20 feet below the top on the outside of the bank. The water then began to ooze out, carrying with it some fine material, and gradually

opening channels for the free passage of water. Then, when the reservoir was suddenly emptied, the saturated bank on the inside was so tenacious that the water in the bank could not escape rapidly enough and the whole mass slid down. The other case was that of a service-reservoir at Portland, Maine. The reservoir-bank, 40 feet high, had been constructed with the greatest care under the direction of an experienced engineer. The inner slopes of $1\frac{1}{2}$ to 1 were covered with clay puddle, on which was laid a bed of broken stone faced with a pavement of granite blocks. On the 5th August, 1893, a section of the embankment at one corner of the reservoir gave way in consequence of the water following a line of pipe laid through the bank, making a gap 60 feet wide at the top and 20 feet wide at the bottom, emptying the reservoir in less than an hour. Shortly after the reservoir was thus emptied, the paving on two sides slid down into the reservoir. In this case also the lining was insufficient to resist the thrust of the saturated clay puddle behind the wall. The immediate cause of the sliding of the bank in both instances seems to have been the presence of a mass of retentive material on the inner slope, which had been intended to prevent the penetration of water, but which had become saturated and caused the mass to slide, instead of draining out gradually as would have been the case with sand or ballast. It seemed likely that such accidents as these might be prevented by adopting the course recommended by Col. Jacob for the toe of the outer slope of the Jeypore dam, and making the exterior of the embankment on both sides of sand for 5 feet overlaid by broken stone carrying a pavement of large rubble. To prevent the percolation of water through an embankment, a thin core wall of concrete or rubble masonry was preferable to a wall of puddle. Mr. Croes.

Mr. T. P. S. Crosthwait congratulated the Gaikwar of Baroda on having given a constant supply of water to his capital, and Mr. Jaganath Sadasewjee on the execution of the work. A gravitation scheme had been adopted by the Author, and in that he was, no doubt, much influenced by the caste prejudice against pumping. That prejudice had been overcome in Calcutta and elsewhere, but it proved to be very strong in Baroda, and probably it was not as easy to combat a prejudice of that kind in a native State as in one under British rule. Mr. Crosthwait, having been engaged, in 1878, in the preparation of a scheme for the water-supply of Baroda, had not seen his way to recommend gravitation, owing to the country around the city consisting of a flat plain intersected by water-courses, of which the beds were 30 to 90 feet below the surface of the plain, the banks being nearly vertical and composed Mr. T. P. S. Crosthwait.

Mr. T. P. S. Crosthwait. of dry sandy clay, with black cotton soil near the surface. Borings showed this to be the general condition, and he had found that any reservoir would have to be very shallow and of large area, from which there would be great loss by evaporation, considerable loss by absorption, and also leakage. The reservoir which has been formed appears to have a general depth of only about 8 feet when full. The leakage under the embankment was first noticed in 1890, and some precautional works were carried out, the leakage increasing to 49,000 cubic feet daily in 1891 and nearly double that quantity in 1892. These quantities had been observed in October, at the end of the Monsoon, when it might be presumed that the reservoir was full or nearly so. It would be interesting to know if the leakage had further increased in 1893, and whether the water in the wells of the neighbouring villages had been raised since the construction of the reservoir. The loss by percolation was not surprising, as the puddle-trench did not appear to have been carried deep enough—not as deep as the beds of the two rivers crossed by the embankment. The average of five years, observations showed the discharge of the Surya river to have been only 18·4 per cent. of the rainfall, which alone showed how porous the subsoil must be.

It had been assumed by the Author that the evaporation from the reservoir would be 72 inches annually. As the general depth of the reservoir was only 96 inches, there would be a large deduction from the storage on that account. The shallow nature of the reservoir would be favourable to the growth of vegetable matter, and as the water would be tepid, the Author had wisely introduced a system of purification and aëration, as well as works of filtration.

Mr. J T. Fanning. Mr. J. T. Fanning, referring to the behaviour of the siphon pipes of the Tansa works, as regards expansion and contraction, had observed that large cylindrical plate-iron conduits, resting horizontally on trestles, when used to convey water from a canal on one side to the canal on the other side of a valley, expanded and contracted daily with the variations of temperature, so far as to require expansion joints at each end of the tube. In the construction of some American water-power works, he had used wrought-iron tubes to convey water under pressure. At Spokane Falls, Washington, two wrought-iron penstocks of 7 feet diameter and 500 feet in length were used to convey a portion of the water-supply to the turbine wheels. These tubes had an inclination of 1 in 12, and conveyed water to turbine-wheels working under 70 feet head. There was daily expansion and contraction of the pipes during construction. At Great Falls, on the Missouri

river, Montana, there was first laid a plate-iron tube of 9 feet diameter and, approximately, 350 feet in length. This conveyed water to turbines working under 40 feet head. During its construction daily expansion and contraction was observed. Mr. J. T. Fanning.

At Austin in Texas, on the Colorado river, eight pipes of 9 feet diameter and 290 feet in length, with branches of 7 feet diameter to the turbines, were laid to convey water to wheels in the power-house to work under 60 feet head, and similar daily expansion and contraction was observed.

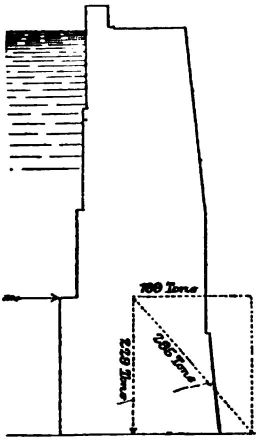
Recently, in the development of the water-power at Great Falls in Montana, a wrought-iron tube of 10·5 feet diameter and 420 feet in length had been constructed for conveying water to two turbines of 900 HP. each, working under 40 feet head. In this case the building of the tube in place was commenced at the masonry of the inlet valve-house, and was carried forward from this point a distance of 350 feet, with an inclination of 1 in 100, toward the power-house. This tube was exposed during construction to all changes of temperature. The movement at the free end of the 350-foot section, before it was connected to the lower section, was on some days found to be fully 2 inches, or 0·000476 of its length, longitudinally, between midnight and midday. The differences in temperature of the upper portion of this tube, exposed to the sun, was probably not less than 70° Fahr. between midnight and midday. The weight of this piece was about 70 tons, and its cross-section was about 100 square inches. It was built first upon blocks of timber without bearings upon the earth, until it was riveted and tested under pressure. If the coefficient of friction of the plate-iron upon the bearings of wood was as much as 0·60, then the stresses of tension and compression in the tube near its fixed end may have been approximately 42 tons, or 0·42 ton per square inch of section, in addition to the stress required to draw the 70 tons weight 2 inches up the incline in, say, eight hours. These tensile and compressive stresses were acting on the masonry of the valve-house in which the higher end of the metal cylinder was anchored fast. These considerations led to the bedding and covering of the pipe at the earliest convenient time. Anticipating some expansion and contraction in the power-house, where the branches from the main pipe led to the turbines, he had deemed it prudent to use an expansion joint near the turbine branches. Since this tube had been completed, bedded, and filled with water, no movement due to expansion or contraction had been observed. Experience with the other tubes at Great Falls and Spokane also

Mr. J. T. Fanning. indicated that there was no apparent daily movement due to expansion or contraction after the tubes had been bedded, covered, and filled with water. The friction between the iron and the earth, when the tube contained its load of water, resisted the longitudinal movement and forced the fibres of the iron to yield all along the pipe to the stresses induced by changes of temperature. If, in the exposed Tansa cast-iron pipes, the expansion in each 12-foot length was approximately 0·0000062 of the length for every degree Fahrenheit of change of temperature, as suggested by experiments, and the daily range of temperature at any time reached 70°, as seemed probable, then there was a tendency to extension equal to approximately $\frac{1}{16}$ inch in each lineal foot, with probably a compressive stress of $2\frac{1}{3}$ tons per square inch of section of the pipe-wall in those pipes which lay horizontally, increased in the case of pipes on the inclines. The stresses of complex expansion and contraction had to be resisted by the pipe-joints as well as by the metal of the castings, and the effects upon the joints must be very severe, and perhaps occasionally cumulative by the combined movement of several 12-foot lengths of pipe. He took additional interest in the behaviour of these exposed pipes since observing that the drawing for the pipe-joints was in form and dimensions identical with a design published by him about fifteen years ago for pipes of that diameter.

Mr. George Farren. Mr. George Farren remarked that, in dealing with the subject of masonry dams, the principles involved in the construction of dams arched in plan ought to occupy an important place. In certain circumstances and under peculiar conditions, the arched form might be introduced as a valuable adjunct to dam building. There were only three dams of considerable magnitude which were incompetent to resist by their gravitational stability the thrust of the waters impounded by them, and which therefore depended for stability upon their arched form. There were the Zola dam, *Fig. 18* (constructed by the father of the well-known novelist), in Provence, and the Sweetwater and Bear Valley dams in California, *Figs. 19* and *20*. The resultants of the stresses, reservoir full, in the Sweetwater dam fell well in the outer third of the section, showing that tensile stress must occur at the inner toe; while in the Zola and Bear Valley dams the secants exceeded their bases altogether, indicating that the arched structure was operating as an arch. Some years ago he had visited the Furens dam, passing through the Gouffre d'Enfer, and had also inspected the Chartrain and Roanne dams. The Furens, and, indeed, most of the French

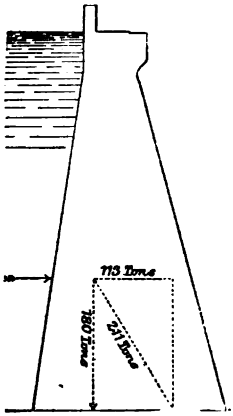
dams, were arched in plan, though the curvature added nothing to their stability, as the direction of the lines of the secants would show. The completed dams visited by him did not appear to be quite water-tight, but in the new Roanne dam, which was probably now about finished, the inner face was being rendered with a mixture of cement and sand in equal parts, laid on about $1\frac{1}{4}$ inch in thickness. As soon as it had set, which occupied about five hours, the water was allowed to rise and cover it, and this treatment appeared to be satisfactory. It would be observed that the work was on the inner face, where it was important to ensure water-tightness in such structures. He had been informed by Mr. George Farren.

Fig. 18.



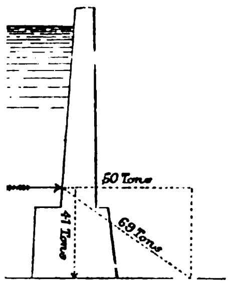
Zola Dam.

Fig. 19.



Sweetwater Dam.

Fig. 20.



Bear Valley Dam.

Scale, 1 Inch = 50 Feet.

French engineers that the Zola dam had done its work well up to the present time, and that they were perfectly satisfied with it. The Bear Valley dam was startling in its boldness, and the stresses occurring in it were very high. He would not care to reside in any town situated immediately below it. It had been reported that it was proposed to erect a stronger and higher dam somewhat below it, and so to relieve the strain in the existing wall, and that seemed a very prudent step to take. In cases where there were no houses below it, such a light structure might be allowed, and the complete and sudden ruin which would attend its failure might be contemplated without alarm; for

Mr. George Farren. example, in a mining district, where the dam was carefully watched by those interested in it. But he thought it would be imprudent to build such a wall above a town or populous district, whose inhabitants could not hurriedly take measures to secure their safety in the event of an accident. He would here compare the Furens, the Zola, the Sweetwater and the Bear Valley dams, in respect of their relative cost, with similar sections below the water-level, cut off from the Furens dam. This comparison would be made in tons weight per middle section for a length of one foot, and the cost might average about £1 per ton.

	Feet deep.	Tons.		Tons.
Actual weight of the Zola dam, say	120	229	Furens	370
Actual weight of the Sweetwater dam . . . . . "	98	180	"	264
Actual weight of the Bear Valley dam . . . . . "	62	41	"	126

showing the relative savings in tons weight which would be gained by the arched form of construction over the gravity type. The savings in money would, however, not be anything like so much, on account of the necessary dressing of the masonry in the arched dams.

A totally different method of comparison might have been adopted by reducing these extraordinary dams to the linear dimensions of the Furens dam, increasing their heights and thickness in the same relation. The objection to that course was that it was uncertain how their respective designers would have modified the sections for such increased height. With a water-pressure of 330 tons, maintaining in each dam the existing condition as to the incidence of the resultant stress, reservoir full, the weights of the several sections per lineal foot of dam were, for the Furens, 740 tons; for the Zola, 513 tons; for the Sweetwater, 500 tons; and for the Bear Valley, 270 tons. He would point out that by reducing all four dams to the same dimension as regarded height, whilst preserving their relative form, the lines showing the direction of the stresses and their resultants would not be disturbed, though of course the stresses would be very different from those in the original forms, as the stress due to the water-pressure was the same in all four. The resultants of the stresses would be, for the Furens dam, 830 tons; for the Zola dam, 552 tons; for the Sweetwater dam, 626 tons; and for the Bear Valley dam, 463 tons.

It was worthy of notice that two of the engineers of the proposed Quaker bridge dam were both opposed to arched dams, and had recommended a straight gravity dam; one of them had

even worked out the problem of the Quaker bridge dam—providing that the stress should not exceed 16 tons per square foot—and had shown that the sectional area of an arched dam about 277 feet high would be more than double that of a gravity dam. On the other hand, other American engineers had strongly objected to a straight dam at Quaker bridge, and recommended a dam curved to a 1,200 feet radius, which, however, was practically a gravity dam, their principal reason being that in case of the yielding of the masonry in a straight gravity dam, "the curved form given to it might prove of advantage," and that "the curved form better accommodates itself to changes of volume due to changes of temperature." Mr. George Farren.

Mr. C. F. FINDLAY mentioned that, on the question of the elasticity of masonry and concrete in relation to the theory of dams, it might not be superfluous to state that even if it were possible to use absolutely homogeneous material, and the modulus of elasticity of that material were known with the utmost precision, engineers would still be at an immeasurable distance from any rigorous determination of the distribution of stress in a body of irregular figure, such as a masonry dam. Mr. C. F. Findlay.

There were two assumptions universally made in the theory of the subject, and almost universally passed over in silence (as by Professor Kreuter) so that they were liable to be mistaken for physical truths. These assumptions were: (1) That all the internal stresses acted in planes normal to the face of the dam; and (2) That in any such plane, if a horizontal section of the dam were taken, the resultant pressure was distributed across the section as a uniformly varying pressure. The sanction for these assumptions was to be found in comparing the deductions made from them with the results of study of actual structures, and not in any scientific theory of elastic solids. Professor Kreuter was in no sense responsible for the hypotheses with which his Paper commenced. He had simply taken them as he had found them, representing the accepted basis of current practice, and had limited his Paper to showing how the form of dam to which they logically give rise could best be arrived at.

The Paper introduced a new, more complete and more direct method, but a few points might be noticed on which it appeared to him that the Author had not done justice to the value and completeness of his own proposals. On p. 66 it was said: "the simplest among appropriate cross-sections . . . is a right-angled triangle with vertical inner face." Now a vertical face was no simpler than a battered face, but, fortunately, there might be found a better

Mr. C. F. Findlay. reason for the verticality of the inner face, viz., that the vertical inner face gave the smallest triangular cross-section consistent with the conditions assumed, when the masonry (as in almost all practical cases) had at least double the specific weight of water. If masonry were used of a less specific weight, maximum economy would be realised in giving a batter to the wall of 1 in $\frac{2 - \rho}{\sqrt{4 + \rho^2}}$, where ρ was the ratio of the specific weights of masonry and water. The only reason why it was not permissible to give to the inner face an equal batter inwards (or overhang) with masonry of ordinary weight, was that the line of pressures, reservoir empty, would then fall outside the middle third. In this connection, it might be suggested, whether, where there was no provision for emptying the reservoir, the case of the reservoir empty need be considered? In the Tansa dam, no doubt this had been fully weighed, but, where a reservoir could only be emptied by the destruction of the dam, it would seem advisable to obtain any economy to be realised by neglecting such a condition.

Next, as to the superstructure, it was not essential to the Author's theory to assume a rectangular block of masonry "equivalent" to the actual superstructure; and in practice a block "equivalent" in his sense would not coincide in height and width at once with the dimensions already fixed by the actual superstructure. It would have been better, therefore, to assume merely a certain weight of superstructure with a line of action not necessarily coinciding with the middle of the width. In the same connection, it seemed a superfluous assumption to take the top-water level as the level of the top of the dam. It might be appreciably higher or lower, and the theory in no way required such an assumption. Next, coming to portion No. (2), an assumption was made "convenient . . . and having the further advantage of simplifying the calculations," that the height of the trapezoidal portion should be equal to that of the superstructure. This resulted in disfiguring the inner face of the cross-section with a re-entering angle. No engineer would accept such a form, and it would have been better to have assumed a vertical inner face for this portion, and to have let the height of it follow as a consequence, instead of being arbitrarily assumed. The resulting height would have been greater than that in the figure, but quite unobjectionable.

Next, attention might be called to a remark, p. 71, that "if the slope of the outer face is to be allowed for in the limiting pressure (as is done by Rankine)," etc. This point seemed to him much

too important to be dismissed as a matter of taste. Judging by the fact that the batter of the face was ignored in the succeeding section of the investigation, it seemed that Professor Kreuter did not consider Rankine's argument conclusive, in which opinion many others would probably join; but the fact of a strong opinion existing on the other side on so important a question demanded that reasons should be assigned for whatever course was adopted. Lastly, as to the portion No. (4) which entailed the most serious part of the investigation and related to the heaviest part of the dam, the Author said: "an investigation which it is needless to reproduce here shows that," etc. Now, as the course which the Author had adopted was only one of several which were open, and as other writers had adopted a different course, he could not concur in the opinion that it was needless to reproduce the investigation in question. Its general nature should, at any rate, have been sketched so that others might be able to find for themselves the same justification which the Author had found for following that course. What the Author had done was to make the pressure at the inner face, reservoir empty, constantly equal to the maximum limit, and that at the outer face zero. Then (vertical pressures only being considered) the pressure at the outer faee with the reservoir full would be always less than the maximum limit, and that at the inner face would always have a positive value. He believed that the conditions laid down would be more economically complied with by making the pressure at the inner face, reservoir empty, and that at the outer face, reservoir full, both equal to the maximum, and keeping the two lines of pressure always equidistant from the centre of the dam. The equations of condition for this arrangement were not simple but were soluble. For the purpose of design, however, he would prefer to use a graphic method throughout. While the analytical method was invaluable in deciding upon the process and in giving a general view of its results, it was inadequate to the complete determination of the cross-section, as might be seen in the treatment of portions (3) and (4); and the whole of the analytical expressions could be translated into graphic methods, which, at any rate, as a check on the calculations, should in practice be applied. Mr. C. F. Findlay.

Professor PH. FORCHHEIMER, of Aachen, remarked that whilst in designing dams the weight of the masonry and water were considered, the uplift and the influence of varying temperature did not appear as elements of the calculation. When water penetrated through masonry it naturally exercised a tendency to lift the superincumbent mass, which was especially manifested at the Prof. Ph. Forchheimer.

Prof. Ph. Forchheimer.

mortar joints. It might be assumed that water flowed in approximately horizontal lines through the masonry; and, as its velocity was constant, it suffered a uniform loss of pressure. This pressure, multiplied by the volume of the interstices per unit volume of the mortar, which might amount to one-fourth, would give the uplift. The safest plan was to make the inner face of the dam as water-tight as possible. In a dam lately built at Remscheid, Westphalia, by Professor Intze of Aachen, the inner face was plastered, and then rendered over with two coats of asphalt. Greater security might be obtained by draining a wall so constructed close to the inner face. Of even more importance than uplift in dams was the movement due to variation of temperature, especially in countries subject to climatic extremes. It had been observed by Professor Intze that, at the middle of the crest of the dam alluded to, which was 82 feet in height, a backwards and forwards movement amounting to $1\frac{1}{16}$ inch occurred during the filling and emptying of the reservoir, and that the movement due to temperature was almost as great as this. The latter was due, less to the temperature of the air, than to direct solar radiation. The crest of this dam was 460 feet long, and was arched with a radius of 420 feet. One side of it was exposed to the sun longer than the other; and the more exposed part moved to and fro $\frac{7}{8}$ inch, in the course of the year, whilst the other part moved only $\frac{1}{8}$ inch the crest expanding $\frac{1}{9,000}$ of its length, or $\frac{5}{8}$ inch. In arched dams such movements did no harm, but in straight dams these phenomena were objectionable. As dams were usually built during the warmer seasons of the year, the masonry had a tendency to contract in the colder weather. In a curved dam this can take place by movement of the structure without cracking, but not in a straight dam. Adopting Adie's coefficients of linear expansion, a reduction of 10° Centigrade in temperature would produce contraction to the extent of $\frac{1}{10,000}$ of the length of the wall. The data concerning the modulus of elasticity of stone was scanty and unreliable; but it appeared generally to be between 1,400,000 and 2,800,000 lbs. per square inch. That of mortar, according to Hartig,[1] was higher. If the temperature of masonry was lowered 10° Centigrade, and it was not free to contract, tension amounting to between 140 lbs. and 280 lbs. per square inch was set up, which was greater than the mortar would stand. When a wall became warmer than it was at the time of its construction it formed a compressed beam; but, owing to the considerable

[1] Civilingenieur, 1893, p. 472.

length of dams in relation to their thickness, and the high compressive strength of building materials, there was seldom danger of fracture from expansion. That a straight, or almost straight wall, incurred considerable danger of fracture was shown by practical experience. The dams of Habra, Grands-Cheurfas, and Sig in Algiers had broken, and in that of Hamiz a tear had occurred during the first filling. The Habra dam broke in December, and the Grands-Cheurfas and Sig dams gave way in the month of February. The Beetaloo[1] dam in Australia had also developed a crack $\frac{1}{8}$ inch wide in the middle of the winter, without any apparent cause. The Mouche dam, Haute Marne, a structure 1,346 feet long and about 100 feet high, exhibited clearly the dangers attending straight dams. In the winter of 1890–91, when the temperature varied between $-10°$ Centigrade and $-20°$ Centigrade, and the water-surface was 10 feet 8 inches below the normal level, seven vertical cracks appeared in the dam, situated at uniform distances of about 160 feet apart. They were widest at the top, and died out about 37 feet below the normal water-level. Their aggregate breadth was $2\frac{7}{8}$ inch. The cracks gradually closed as the temperature rose, and by the end of February, 1891, four of them had completely vanished, whilst the others had perceptibly contracted.[2] In other buildings, also, contraction during cold weather caused cracks. A long quay-wall[3] at Bremen, for instance, developed cracks due to low temperature, which opened between $\frac{1}{8}$ inch and $\frac{1}{3}$ inch in winter and closed to fine hair cracks during summer. Prof. Ph. Forchheimer.

Prof. FRÜHLING, of Dresden, noticed that the Author of the Paper on the Tansa Waterworks attributed the bursting of several 48-inch pipes mainly to an insufficiency of air-valves; but many hydraulic engineers would look for the cause of failure in the bedding of the pipes upon the rock. Prof. Frühling.

In the Baroda Waterworks there was no provision of a third high-level inlet to take the upper portion of the water from the collecting channel. Such an arrangement was desirable, especially as, during the rainy months, this upper supply had held a minimum quantity of solid matter in suspension. The circumstance of the omission of a puddle-wall in the embankment appeared, in spite of the good quality of the materials employed, to have been the primary cause of the percolation of water to the toe of the outer slope of the embankment.

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[1] Minutes of Proceedings Inst. C.E., vol. cxiii. p. 158.

[2] "Reservoirs in the South of France." By Marius Bouvier. Fifth International Congress on Inland Navigation, Paris, 1892.

[3] Zeitschrift des Architekten-und Ingenieur-Vereinszu Hannover, 1889, p. 444.

Prof. Frühling. In the case of the Jeypore Waterworks, percolation was to be expected from the nature of the material of which the dam was constructed, and the impracticability of introducing a puddle-wall. It might, however, be questioned whether, failing the insertion of a puddle-wall, it would not be better to construct the flatter slope on the outer side. In this case illustrated in *Fig. 21* the greater portion of the dam was dry, in contrast to the case of *Fig. 22*, whilst the leakage was less owing to the smaller inclination. In regard to the plan of the service-reservoir, it might be remarked that, as a consequence of the inlet and outlet being placed opposite one another, the water would not, generally speaking, circulate sufficiently freely through the reservoir, whereby the freshness of the water was lost the development of bacteria was favoured.

Fig. 21.

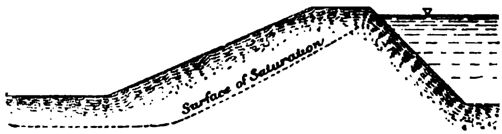

Fig. 22.

Mr. J. H. E. Hart. Mr. J. H. E. Hart presumed that the constructors of the Baroda and Jeypore reservoirs had, to a certain extent, followed the Bombay irrigation standards for earthen dams, introduced by General Fife, R.E. The peculiar feature of his practice was the rejection of the puddle-wall, which was typical of the English system. By depositing the material in thin layers, saturated with water, and carefully consolidated, he, in fact, made the entire dam a mass of puddled earth. A serious mistake had been made in not reaching a clay bottom with the first puddle-trench of the Baroda embankment for the 200 feet between the rivers. It was stated that it had been reached with the second trench all through. But why not with the first? for the difficulties must have been similar in both cases. The second trench was, in his opinion, quite useless; for it was quite unconnected with either the mass of the dam or the other puddled trench. A somewhat indiscriminate use of the terms "sand" and "earth" had been adopted by the Author of

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the third Paper, but the dam described ought to have had something in the shape of a puddle-wall up the middle, for it appeared that it was little more than a filter. The way in which the water falling on the surface of the embankment was disposed of would not answer with good stiff material. The guttering of the slopes was prevented in the irrigation dams by a thick outside casing of mixed black soil, and decomposed trap and gravel. The questions of the discharge from catchment-areas, and the evaporation from reservoir-surfaces, had engaged the attention of Indian engineers, as well as those of other countries. In the nature of things no definite answer could be given to these questions, because each catchment must necessarily have its own coefficients. Surprise might well have been expressed at the large estimated discharge of the Tansa waste-weir—25,000 cubic feet per second—but the maximum discharge provided for over the waste-weirs of certain reservoirs in the Deccan amounted to 200,000 cubic feet per second in two instances. At Lake Fife, with a catchment area of 196 square miles and an estimated discharge of ½ inch per hour, the waste-weir was calculated to discharge 62,000 cubic feet per second; and at Bhatghur, with ⅝ inch discharge per hour from 128 square miles, and the discharge provided for was 57,000 cubic feet per second. He might here direct attention to the large masonry and concrete dam at Bhatghur, which formed the impounding reservoir of the Nira Canal, designed by Mr. J. E. Whiting, M. Inst. C.E., and now complete, or nearly so. Its height was 127 feet above the lowest foundations, and 103 feet above the river-bed. It was 3,020 feet long, with 810 additional feet of waste-weirs. The upper part was built of concrete faced with masonry, the lower part being solid rubble; and it carried a roadway 11 feet wide along the top. The section of the dam was designed by Mr. Whiting, and all the calculations were worked out by Mr. A. Hill, Assoc. M. Inst. C.E., who was afterwards Mr. Clerke's assistant on the Tansa works. The curve adopted for the face was a catenary; but the wall was actually built in a series of batters. The three primary conditions of the design were, 1st, the intensity of the vertical pressure was nowhere to exceed 120 lbs. per square inch; 2ndly, the resultant pressures were to fall within the middle third of the section; and, 3rdly, the average weight of the material was to be assumed at 160 lbs. per cubic foot. The use of concrete in the section was only permitted where the pressure was calculated not to exceed 60 lbs. on a square inch, which gave a factor of safety of between 6 and 7. With reference to the movement of dams under the pressure of water,

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he had found, after prolonged and careful observations with theodolites, the supposed deflection of the Kurruckwasla dam—Lake Fife—was a myth. Indeed, at times, it seemed to move in a direction contrary to the pressure. The observed anomalies were considered to have depended on the temperature and on refraction. Finding that instrumental observations were unreliable, he had arranged with Mr. Whiting, when building the dam at Bhatghur, to leave a vertical pipe in the masonry through which a plumb-line could be dropped, so that the movements, if any, could be directly measured.

Col. J. O. Hasted.

Col. J. O. Hasted, late R.E., supposed that, as the three reservoirs which had been described are accomplished facts, and were fulfilling the duty for which they were intended, they must be considered successful examples of their class, but it was rather startling to find that in two of them the embankments had been formed simply of the adjacent soil thrown in a heap on the ground.

In the case of the Jeypore reservoir the embankment was of sand, without any core-wall, resting on sand and mud, and, when full, the water would stand 40 feet deep against the embankment. It seemed to be a bold undertaking, and he noticed that it had been considered desirable to take precautions in case of leakage appearing at the toe of the outer slope of the embankment.

He had, during a long service in the Irrigation Department in Madras, been obliged to make embankments of sand on a smaller scale, and well remembered one instance, after the cyclone of 1864 at Masulipatam, when it was his duty to excavate a channel from a navigable canal some 3 miles from the town to bring in fresh water, all the wells having been ruined. That channel had to cross hollows where the water was held up by embankments, and the only material available was sand. It was almost a matter of life and death, so, working night and day, the water was brought in; but the sand embankments caused endless trouble. They were continually giving way, and for some time had to be watched day and night, work-people being kept ready to stop breaches in them until the river-water, which was highly charged with silt, had in effect puddled the inner slopes.

That sand might be compressed by weight until it became capable of withstanding considerable lateral pressure, he was well aware. The great weir across the Kistna river at Bezwadah was a masonry wall about $\frac{3}{4}$ mile long and 14 feet high, resting on wells 6 feet deep sunk in the sandy bed of the river. On the down-stream side of the wall there was an enormous talus or

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apron, over 200 feet in width, formed of blocks of rough stone, its surface being sloped at about 1 in 20. At one time, owing to some particular set of the river, the sandy bed was eroded above the weir, until there was a hole 30 to 40 feet deep, and below the apron there was another hole 90 feet deep; and as the flood subsided there was a difference of level between the water above and below the weir of 10 to 20 feet, but the weir never moved, the weight of the stone having compressed the sand so as to render it quite firm.

In the case of the Baroda reservoir the embankment had no puddle-wall, as the material employed in its construction was of good quality, which, he supposed, meant soil with some admixture of clay, but there was a puddle-wall in the ground below.

No doubt the old reservoirs constructed by the natives of India, called in the Madras Presidency, where they numbered some 39,000, tanks, were made by simply throwing up the earth to form a bank; but those banks frequently gave way, and in some of the larger ones, that had stood for many years, he had known great trouble arise from leakage. The toe of the outer slope was apt to become saturated, and the slope, and possibly more of the embankment, sat down. The only remedy was to find where the water came from, and to lead it away in rough stone drains to a distance before filling in again with earth.

In reservoirs with earthen embankments there was always risk of leakage, and therefore a good puddle-wall, carried well down through the subsoil, was desirable. In India it had sometimes been the practice to form the puddle-wall along the face of the front slope under the revetment. This, however, was not a good arrangement in a tropical climate, as when the water fell in the reservoir, the puddle was liable to dry and crack. In some cases, when the puddle-wall rested on soil of such a nature that it was possible water might creep through, it was desirable to lay rough stone drains to convey the water to a distance in rear of the embankment.

He would also, with reference to the earthen embankment reservoirs, remark on the long front slopes employed—in one case 3 to 1, and in the other case 4 to 1. He preferred a steeper slope in front with a long slope in rear for this reason, if the water-spread was considerable the waves formed by wind so easily ran up a flat slope. A case in point was that of the Red Hills reservoir, which supplied Madras with water, and was breached in a cyclone in 1884. The front slope there was 1 to 1, and the revetment was formed of fairly smooth blocks of stone. When the

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gale was blowing at an angle of about 25° to the embankment, the water ran along the face, rising more than 9 feet, and overtopped the parapet-wall. When the reservoir was reconstructed he had provided for the lower layer of stone in the vertical parapet-wall being projected as a string-course purposely to break waves running up the front slope.

With regard to the disposal of the rainfall which fell on the Jeypore embankment, the water which fell on the top was allowed to soak into the bank, which thus became a huge sponge. In a dry climate very likely this was all right, but he could not recommend such an arrangement in any locality near the coast in India, where very heavy rainfall sometimes occurred; when the sponge became surcharged the result would be disastrous. When the rear slope of the embankment was well grassed over, the rain-water might safely run off the surface, if it was conducted away from the toe of the slope, but until it became so grassed over it would be advisable to form channels to carry off the rainfall.

The material of which the Tansa dam had been constructed was uncoursed rubble masonry set in kunkur limestone mortar. He would say the smaller the stone the better. With the very considerable pressures in the dam, anything which tended to uneven settlement was to be avoided, and the mass should be as homogeneous as possible. With this view he had advocated the use of concrete in the construction of the dam, 155 feet high, at the Periyar Reservoir, in South India, which was now in progress.

Mr. Clemens Herschel.

Mr. Clemens Herschel desired to comment on two points suggested by the interesting group of Papers under discussion. One was the substitution of riveted, or lap or butt-welded, or seamless drawn, or rolled steel pipes for cast-iron pipes; the other was on the method of keeping the records of the flow off a catchment area. As regards the former, it had been demonstrated by practice in the States west of the Rocky Mountains, in the United States, that cast-iron could not compete with wrought-iron or with steel pipes. This had been the fact for the past thirty years, unless in exceptional cases. Scarcely any main water-pipes were now laid, or had been laid for thirty years past, on the Pacific Slope, other than some form of wrought-iron or steel pipes. This had been due to lack of coal and of iron ore in those States, rendering necessary the importation of iron and steel; and the weight of cast-iron pipe compared with that of steel pipe of equal capacity and strength, had put the former out of the competition. It might be thought that in India, similarly situated as regards importation of cast-iron pipe, which was worth £5 15*s.* 8*d.* to £5 19*s.* 11*d.* per ton, delivered on the wharf, that material would

be equally out of the race. With another such case of a main to be laid on a stretch of country reaching 50 miles into the interior, no small economy could be effected by the employment of steel pipes. The cost of the 17½ miles of siphons did not appear, but the pipe on the wharf was about $15·25 per lineal foot, assuming an average of the three prices stated; and experience with riveted pipe led him to say that such siphons could be laid and completed for less than the cast-iron pipe cost on the wharf. Clearly, India should offer a great field for riveted or other steel pipes. Steel pipe was a source of economy, not only in exceptional places like California, Nevada, and the East Indies, but also nearer home. The same was true in the eastern states of the United States, and in England. Only the diameter of the pipe appeared, in these places, to limit the field within which either kind of pipe might prevail over the other. Thus it has been established in the States last mentioned, at prices which have ruled for the past two or three years, that steel pipe was more economical than cast-iron pipe down to about 36 inches or 34 inches in diameter. The engineers of Rochester, N.Y., who had laid riveted 24-inch and 36-inch iron pipes in 1874, were now laying a 36-inch riveted steel conduit, some 20 miles long. The supply of Newark, N.J. (and at the same time that of the towns of Montclair and West Orange), was conveyed by a 48-inch and 36-inch riveted steel conduit, 26 miles long, laid in 1889–92.[1] In the manufacture of lap-welded steel and iron pipes, and of butt-welded (electrically (?)) seamless drawn or seamless rolled steel pipe, improvements in manufacture, or reduction in the price of the product, were constantly taking place. Thus lap-welded pipes in 20-foot lengths, and 24 inches in diameter, were already in the market. Tubes of 4 feet diameter, butt-welded, were in use for high-pressure steam-boilers, though still too expensive for water-pipe. And it would, no doubt, only require an order for 10,000 tons to start the manufacture of some of the other forms of pipe that had been named. There was a fair prospect that, before long, steel tubes of any desired strength and diameter, 25 to 30 feet long, might be purchased in any desired quantity, at a price that would make them the most desirable class of pipe for conveying water long distances. It was not unlikely that seamless rolled steel tubes might be furnished at little, if any, over the price of steel plates. Mr. Clemens Herschel.

As regarded the second point—a uniform method of keeping the records of the discharge from a catchment-area—English engineers

[1] Minutes of Proceedings Inst. C.E., vol. cxiv. p. 418.

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had great opportunities in the course of their practice, extending into all corners of the earth. Those opportunities appeared however, to have been in this respect neglected. The necessities of construction and lack of control of works after their completion, to some extent explained the paucity of records of stream-discharge. It seemed, however, that more might be done and recorded, if a uniform system of keeping such records were adopted. No gaugings of the discharge of the Tansa river accompanied the first Paper, the yearly discharge for five years was given in the second Paper, and the third Paper contained the mere statement that only one-sixtieth part of the rainfall on the catchment-area had been shown to reach the reservoir. The weight usually given to irrelevant records of rainfall in engineering Papers of this sort, afforded perhaps a clue to the reason why the important subject of stream-flow had been so neglected, and a consideration of the greater ease with which rainfall records might be arrived at, compared with the measurement of daily stream-flow, would probably solve the question. Civil engineers were under a moral obligation to present in engineering Papers, useful facts to their professional brethren, and if either stream-flow or rainfall was to be omitted, they should surely omit the rainfall, which might be safely relegated to the weather reports, or might be studied in connection with other classes of engineering Papers, such as, for example, treat of sewerage-works. What engineers wanted for water-supply purposes was the daily stream-flow, measured over a long series of years; and no harm would accrue if the matter of rainfall were ignored in such works entirely. It had, indeed, but a mildly scientific interest, and its ratio to the discharge of a stream, possessed only the interest that appertained to matters of scientific curiosity. It was of no service in the design of works for water-supply, or of irrigation works, which a knowledge of the stream-flow alone would not better render. As for the relation of the rainfall during a particular month to the stream-flow during that month, it presented an effect coupled with only one of the causes that produced it, such as necessarily gave results palpably absurd. Work of this kind brought the art of the civil engineer into disrepute with other practical men, and it should be abandoned for that reason, if for no other. To demonstrate the unscientific, untruthful and utterly useless character of such relations between rainfall in any one month, and stream-flow in the same month, it was only necessary to look at, say, the Minutes of Proceedings Inst. C.E., vol. cxiii, p. 321, where it was stated that a certain river in India discharged during the rainless season about ten times as much rain as fell on its drainage-area, due to the river being fed

by rain that had fallen months previously. Discharges of 150 to 250 per cent. above the rainfall were of annual recurrence in the body of such tables during the winter months in cold climates (due to the streams discharging melted snow and ice which had fallen months previously). Again, such relations had a great range in the same place from month to month, and during the same month in different years. Years having the same rainfall yielded different portions of it, according to the sequence and the distribution of the rainfall throughout the year. Knowledge of the rainfall during any one year, and of the portion of it yielded by the river, was therefore no valuable guide as to the stream-flow during any other year, or even during the same calendar month in any other year. That was true on one river, and even on the same portion of the same river. It was also true for different rivers in different countries, and even for different portions of the same river in any one country. Again, to arrive at these relative yields, it was first necessary to know the discharge of a river. But inasmuch as the discharge of the river was the thing sought, why not rest content when it was found? An analysis of the flow of any stream, and of its proportional yield of rainfall, might be useful some day when studied in conjunction with such data for other rivers, and as teaching the effects of steepness of slope, and of quality of the soil, on the yield of streams; but such refinements applied to the remote future, while to-day, the actual yield and discharge of rivers in general formed information much needed by engineers, whenever an impounding reservoir or other water-supply work had to be constructed. With these ideas in mind, the writer desired to suggest for general adoption among civil engineers, a plan of reporting the discharge from catchment-areas, somewhat as follows: Mr. Clemens Herschel.

For areas of catchment-surface and of reservoir surface: square miles.
For discharges of rivers or streams during one month or shorter period: cubic feet per second.
For specific discharges of drainage areas: cubic feet per second per square mile.
For volumes contained in the reservoir: cubic feet.
For rainfall: inches in height.
For yield of drainage-area, when it was to be represented in bulk, as for a year or month: inches in height (considered as covering the whole area on which the rain fell); or else, in cubic feet per second, on the average.

Mr. Clemens Herschel.

Experience had taught him the propriety and convenience of the suggested units for use by civil engineers. They were drawn up for the use of English-speaking engineers only. It must commend itself to them that some uniformity of action should be adopted, so as to make convenient, and ensure the use, by one engineer, of the work of another. So long as such arithmetical vagaries as reporting the discharge of rivers in "tons per 24 hours," annual rainfall on a catchment-area in gallons, discharge in cubic feet per second per 1,000 acres, and so forth, were indulged in, the information afforded was more apt to be passed by than to be treasured up and used. As to the data to be reported, he would suggest:

(*a*) The cubic feet per second flowing in the river, or which would flow in it were none withheld in the reservoir, daily or as often as could be readily arrived at. This might be called the "natural discharge" of the river under consideration.

(*b*) The drainage-area in square miles above the point of gauging the river; the character of the drainage-area as regards slopes, and the quality of the soil.

(*c*) The available volume of the storage-reservoirs or reservoirs, *i.e.*, between top water and extreme low water, in cubic feet.

(*d*) The area of the water-surface of the reservoir or reservoirs, in square miles.

(*e*) The rainfall in inches, monthly, or daily.

(*f*) Natural discharge of the river, monthly or annually, to be expressed in inches of rainfall on the catchment-area; or to be given as an average in cubic feet per second per square mile of catchment-area.[1]

Mr. A. Hill.

Mr. A. Hill stated that in India catchment-areas varied much in regard to the amount and certainty of rainfall. In all cases it was desirable that the assured annual supply should not be largely in excess of the quantity required, as, otherwise, the reservoir was liable to silt up. The Tansa reservoir was most favourably situated in this respect. At the Bhatgarh reservoir for the Nira Canal, where there was a masonry dam more than 4,000 feet long and impounding a depth of 100 feet of water, the total volume of which exceeded 4,600,000,000 cubic feet, the ordinary annual supply was some six times the volume of the reservoir. To minimize

[1] A set of Tables constructed on the lines indicated, exhibiting the characteristics of the discharge from the drainage-area of the Pequannock river in the State of New Jersey, U.S.A., was furnished by Mr. Herschel in illustration of his remarks, but space did not admit of its reproduction in the Minutes of Proceedings. They may be consulted in the Library.—Sec. Inst. C.E.

silting of the reservoir, a series of fifteen sluices, each 8 feet high and 4 feet wide was constructed near the bottom of the dam. The area of these sluices was sufficient to pass all ordinary floods under a head of less than 30 feet, and as the full depth of the reservoir was 82 feet, the greater portion of it, when these sluices were open, was empty and could not be silted up. The flood-water also remained but a short time in the reservoir, and did not deposit so much silt as they would during a slow passage through the reservoir when full. The sluices were closed in ample time to fill the reservoir, and the river water was then comparatively clear and free from silt, grass and vegetation having grown on the catchment-area during the rains. Such a system of sluices could of course only be adopted where the supply was regular and assured, like that from the western ghauts of India during the monsoon. For all sluices, whatever gearing were adopted for raising the sluice-gate, it was important that it should show the position of the gate; in practice, a simple nut and collar with holding-down bolts at the top of the rod, the top length of rod being screwed to fit the nut, was the best arrangement. The gearing should be set so that the top of the rod was flush with the top of the nut when the shutter was closed, and the position of the shutter was then always known, for if 6 inches of the lifting-screw were visible the shutter was raised 6 inches. With this gearing also the screws could be oiled and cleaned. At the Bhatgarh dam the same uncertainty of rock foundations was experienced as at Tansa; at one place a sheet of sound rock suddenly stopped and a drop of 30 feet had to be excavated to obtain good foundations. This was unexpected, for the trial-pits here were not more than 150 feet apart and showed the same kind of rock at about the same level. The drop in the foundations necessitated a change in the position of the waste-weir. Again, in the breadth of the dam at the lower parts, in one place a fissure some 15 feet deep and 20 feet wide occurred; and what was most expansive of all it was found that apparently sound rock in the river-bed was a slab of rock not bearing evenly on the rock below. This slab varied between 8 feet and 30 feet in thickness, and it was deemed advisable to remove it all for about 250 feet in length and more than 70 feet in breadth. No accurate estimate of the cost of a dam could be framed until the foundations were laid. In constructing a masonry-dam it was very convenient to have a sluice at the bottom of the structure, through which water could be let out for washing sand, for bathing, and for the many other requirements of a large work. The sluice-way could be readily plugged if desired when the work

Mr. A. Hill.

Mr. A. Hill. was finished. At Bhatgarh, such a sluice was built at the level of the bed of the river; and at 16 feet from the inner face the sluice-way was reduced from 6 feet to 4 feet in breadth. It was afterwards built up under a head of 10 feet of water; the leakage through the planks which closed the sluice being collected into a 4-inch pipe fitted with a valve. This 16-foot plug of masonry was now subjected to a head of 100 feet of water and was quite water-tight. The sand was washed in shallow masonry pans 20 feet by 10 feet by 1 foot deep, with a stream of water running through them; and sand with 15 per cent. of silt in it, could be perfectly cleaned by that means. The water-supply was practically unlimited, and from 12 to 15 cubic feet of water was used per cubic foot of sand washed. Sweating or percolation of moisture occurred in all masonry-dams in the Deccan. The trap stone was very dense, and that used at Bhatgarh and Vir weighed 183 lbs. per cubic foot. Nevertheless water could penetrate the stone, and when quarrying blocks in the lower parts of the quarry which had been under water during the rains, water was found in the small cavities common in trap rock when the stones were split under the hammer. In the hot weather the rock did not contain water if it had been exposed some time. In the Paper on the "Designs of Masonry-Dams," the Author had stated, "that the shearing-stresses acting parallel to the layers of the wall were not allowed for." These stresses had also been neglected in calculating the maximum intensity of pressure—in other words, the horizontal thrust of the water was neglected. It had frequently been shown that this course led to serious under-estimating of the pressures. It had been repeatedly pointed out that the whole resultant of the weight of masonry and water-pressure combined must be considered. Again, in calculating the intensities of pressure, horizontal planes or bases had been considered by Prof. Kreuter. He desired to suggest that such a course was not correct, and that a plane at right angles to the neutral axis of the cross-section of the dam should be considered, in other words, the profile of the dam must be considered; and, as Rankine had pointed out, the intensities of pressure were in some way affected by the shape of the cross-section or profile. In selecting the final profiles for a dam, facilities of setting-out and construction should be considered, and it should also be remembered that the upper parts of the dam were the longest, the lower parts being confined to the gorge in the river; a little excess of section in the lower portion was therefore less wasteful than equal excess in the upper portion of the dam. The easiest profile to construct was an arc of a circle, and the

most difficult was the long straight batter. In a long straight batter, if the masons made a mistake, or a template were wrongly set, the error could not be rectified without causing a bulge in the face; on the other hand, if the profile were an arc of a circle, a mistake could be promptly remedied by introducing a chord or a tangent as required. Mr. A. Hill.

Mr. W. R. Hutton, of New York, thought that the profile of the Tansa dam had been based upon the true principles which should govern such designs, with ample margin for safety. The construction also, in masonry of uniform character throughout, and the care taken that no voids should be left in the mass, left nothing to be desired, if the latter condition, most difficult of enforcement with ordinary workmen, had been successfully carried into effect. The expense of excavating the firm but fissured rock down to the solid trap, was justified by the experience at the Bouzey dam (Eastern Canal of France), a portion of which was displaced by sliding forwards on its base—a movement rendered possible, it had been stated, by the reduction of its effective weight by the up-lift of the water in the seamy rock on which the dam stood. It had been justly observed by the Author that the character of the mortar to be used in such works was of the first importance. The experiments of Mr. Bouvier, quoted in the Paper, upon the compression of cubes of mortar, confined on all sides in blocks of wood, did not seem to reproduce the conditions of actual practice. Those of Mr. Gillmore upon short prisms of different heights afforded a better guide in estimating the resistance of the mortar in a wall. In the latter experiments the shorter prisms showed a considerable excess of resistance to compression over cubes of the same material and composition. It should be observed that the greatest pressures were always at one or other face of the masonry, where the mortar was unconfined on one side. The percolations through the Tansa dam were so far of little importance. In this respect it was more fortunate or better built than many other high masonry dams. A calcareous efflorescence on the lower part of the outer face, due to percolation, was frequently found. The greatest care in construction, raking out the joints and filling them, and plastering the upper face of the dam with Portland cement, did not always prevent the occurrence of such leakage. Mr. Bouvier seemed to have feared a slow solution and washing away of the lime in the mortar, particularly in primitive formations, whose waters being very pure had great dissolving-power. A large amount of hydraulic lime in the mortar of impounding-works afforded a guarantee of their stability. Mr. W. R. Hutton.

Mr. W. R. Hutton.

But it was none the less necessary to devise means to prevent further impoverishment by the slow action of percolation. The use of asphalt in this connection had received attention. Asphalt coatings on the slopes of the embankments of some of the French reservoirs had failed. Recent lining with similar material of reservoirs in Colorado and California, U.S., had so far been successful. In building the brick conduit in the High Bridge of the original Croton Waterworks, General George S. Greene had required one course of bricks to be heated and dipped half their width in hot coal-tar, and laid in that material. That conduit had never leaked. The head of water on it was, however, small. The observations made in the Paper upon the discharge of the aqueduct indicated the necessity of most careful description of the surfaces of channels. Bazin's coefficient for his third category was applied by Kutter to walls of dry rubble, and not to a conduit of which one-third the wetted perimeter was of plastered concrete and the remainder of well-pointed rubble.

Prof. Kovatsch.

Professor Kovatsch, of Graz, pointed out that, thousands of years ago, Hindoos, Chinese, Egyptians, and other great peoples, had, instead of expending their powers in river improvements or canalisation, constructed immense impounding reservoirs for irrigation and other purposes. Many recent examples had afforded instances of the necessity for clearly determining the principles and details of construction of such works, and the disastrous results due to false economy. It was only by the progress of scientific investigation and study that accurate methods of construction appropriate to the requirements of the case, had superseded the tentative and haphazard works of earlier times. The Papers were replete with practical information and valuable data; while Professor Kreuter's exposition of the design of masonry dams completed the consideration of the subject alike from the analytical standpoint.

Mr. Graham R. Lynn.

Mr. Graham R. Lynn remarked that the Author of the Paper on the Baroda Waterworks had noted that, as the general material used in the embankment was of good quality, no puddle-wall was constructed in it. Looking to the leakage that had taken place, it seemed, however, a pity that that precaution was not adopted; as the best of the material used had 37½ per cent. of sand in it, and some of the samples taken from the waste-water-course had proved on analysis to have contained as much as 66 per cent. of sand. Lately, analysis had shown the albumenoid ammonia in the lake to have increased much above that given by the Author, and had also proved that there was only a slight diminution of that impurity after the water had passed through the purifiers and filters.

Mr. Graham R. Lynn.

That had led to an investigation of the purification-works. The Anderson purifiers, which were each originally charged with 1½ ton of cast-iron borings and had each been daily replenished with 20 lbs. per day, were found to be empty and entirely devoid of that material. They had no doubt been inoperative by reason of the light borings having been carried through the cylinders by the flow of the water. Steps had since been taken to replace the borings by punchings to avoid the recurrence of such a condition in future.

Mr. A. T. Mackenzie.

Mr. A. T. Mackenzie said that the largest dam made in England of late years was that of Vyrnwy, which the Author of the first Paper had diplomatically described as "magnificent." If the Corporation of Liverpool was satisfied with the extravagant proportions of that dam, no one else need complain, so long as the design was not set up as a standard. The theory of dams was simple and advanced enough. What was wanted to reduce their proportions was faith—in the theory and in the material employed in constructing them. Referring to details, it was true that in the matter of foundations, trial-pits give no satisfactory indications, and, unless the bare rock lay naturally exposed to view, it was impossible to estimate accurately the character and cost of the foundations. The original rock exposed to atmospheric influences gradually disintegrated, and above the bed-rock there was sure to be a layer of boulders with seams that might descend 10 feet to 60 feet. That has been so in the case of the Periyar dam. Whether in the case of the Tansa dam it was essential, particularly on the higher levels, to entirely remove those boulders was another question. At any rate, if they were in the middle, or towards the rear of the dam, it would probably have been sufficient to rake out the joints as far as it was possible to reach, subsequently filling in the seam with mortar. At the Periyar dam, broken stone for concrete was being carried by coolies from a shoot to the work, a distance of over 100 yards, for less than 1*d.* per ton, and under the circumstances of the work no machinery could compete with that. The Tansa dam was said to be practically dry, but a dam of mortar only would probably be that. The Periyar dam was being built with a front rubble masonry wall of varying thickness, never less than 6 feet and sometimes as much as 20 feet thick and a rear wall of similar construction, the heart being of concrete. The concrete laid in 6-inch layers and tested in a block of 1 cubic yard unsupported at the sides, leaked between two layers first under a pressure of 75 lbs. per square inch. In 1-cubic-foot blocks, unsupported at the sides, it did not crush under a load of

Mr. A. T. Mackenzie. 12 tons. The rubble masonry was built of the largest manageable stones, averaging perhaps $1\frac{1}{4}$ cubic foot, and had stood in the course of the work such trials by nature that it had not been thought necessary to incur the expense of testing it.

The question of passing the flood-water was of the greatest interest. The floods at Periyar were occasionally enormous, the maximum known being 120,000 cubic feet per second, so that the matter was more serious even than at Tansa. There was no chance of building the foundations on dry ground, and a year-and-a-half was spent before a stone of the dam was permanently laid in the river-bed. A diversion had been made round the flank which was intended to discharge 2,000 cubic feet per second; but it was always breaking into the foundations, and floods of any magnitude had of course to pass over the entire work. On the whole, hardly any damage had been done to the dam, but enough had been learnt to show the enormous harm that could be done by eddies if they were once formed. The lake already formed comprised a large area, and small floods were absorbed before topping the work; though occasionally a weir left lower on one flank of the dam was submerged for a few hours. The ordinary discharge of the river had always been passed round the flank by vents; which of course entailed a protecting wall and a joint. There was also a guiding wall, a continuation of which formed a channel conveying water to a turbine to drive the stone-breaking and mortar-mixing machinery.

It would be useful to learn the experience of engineers in obtaining sand by mechanical means. The sand at Periyar was dredged from the river-bed, but as the water became deeper the process became more difficult and costly. The earth which covered the syenite of the hills was really disintegrated rock containing a large percentage of silica, which could easily be separated by washing. But there always occurred a considerable portion of alumina which did not dissolve in water for a long while, though it yielded fairly easily to pressure. To get sand from this earth, it had to be washed, then passed through stamps or grinding-mills, and finally washed again. The sand in the river-bed was often covered with a layer of fine silt, and contained vegetable matter and a good many pebbles, though when screened it was of fair quality. Tailings from the stone-breakers had been tried, but, besides being limited in quantity, they contained too much fine dust, too many small bits of stone, and a perceptible quantity of alumina.

Mr. R. Moore. Mr. ROBERT MOORE, of St. Louis, U.S., mentioned that the wisdom

of the precaution taken by the Author in carrying the foundation of the Tansa dam below all veins of soft material in the rock, had been strongly emphasised by a recent event in Texas, when the failure of an important masonry dam, over 60 feet high, during the last summer, had been caused by neglect of that precaution. He had visited the work shortly after the fracture occurred. The water had found its way through soft seams in the foundation, and had exerted almost incredible destructive force. Heavy masses of masonry had been undermined and thick ledges of overlying rock had been completely broken up. In considering the schemes for the water-supply of Bombay and Baroda as set forth in the Papers, nothing struck an American engineer more forcibly than the small provision of water per head of population. For Baroda 25 gallons per head per day had been considered an ample supply, whilst it had been estimated that 50 gallons per head per day were more than would ever be required for Bombay. To Americans, accustomed to a consumption of 75 to 100 gallons per head per day, those figures indicated differences in the scale of living which it would be interesting to investigate in detail. But whatever the cause, the simplification of the problem of water-supply to Indian cities produced thereby were most important. Mr. R. Moore.

Lieut.-General JOHN MULLINS, late R.E., observed that, including additional water, received during the continuance of the rainy season, the quantity of water made available by the Tansa works was, in round numbers, 3,400,000,000 cubic feet, and on that basis the cost of providing storage amounted to about £70·59, or Rs.1,129 per million cubic feet—a rate much in excess of that financially admissible for storage for irrigation. The Tansa site presented favourable conditions as regards water-supply, reliable foundations for the dam, abundance of stone for its construction, and facilities for the disposal of surplus water. It necessitated, however, a great length of dam; and the alignment of its axis, though doubtless determined by the circumstances, was not of the most desirable form, being concave towards the water. He had had occasion to consider the subject of formulas for approximately determining the proportions of masonry dams for reservoirs, and had concluded that a modification of Sir Guilford Molesworth's formula for deducing the width of the base, exclusive of the part appertaining to the inner face batter would be desirable. The modified formula was— Lieut.-General John Mullins.

$$y = b + \sqrt{\frac{0\cdot05\,(x-b)^3}{\lambda + 0\cdot05\,(x-b)}},$$

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in which x = the depth in feet of any horizontal plane below top of dam;

y = offset in feet, from a vertical line through the inner top edge of the dam, to its outer face at any depth x;

b = breadth in feet of the top of dam;

λ = the limit of pressure in tons per square foot.

The minimum limit of b was 7·50 feet, and for a less top breadth than 12 feet a somewhat different form was given to the equation, viz.:—

$$y = b + (12 - b) + \sqrt{\frac{0 \cdot 05\,(x - 12)^3}{\lambda + 0 \cdot 05\,(x - 12)}}.$$

For the base of the inner face batter, the formula was

$$z = \left(\frac{0 \cdot 09\,x}{\lambda}\right)^4$$

in cases in which the reservoir, as at the Tansa site, was not liable to be emptied, and became

$$z = \left(\frac{0 \cdot 0925\,x}{\lambda}\right)^4$$

where that condition did not obtain. It might be interesting to compare the dimensions given by these formulas with those adopted in the design for the Tansa dam, at a few depths (x) below the top.

Values of x.	Tansa Dam.[1]			Mullins' Formulas.		
	y.	z.	$y + z$.	y.	z.	$y + z$.
Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
50	27·50	..	27·50	29·31	0·15	29·46
60	33·00	..	33·00	35·94	0·30	36·24
90	56·00	0·60	56·60	58·13	1·56	59·69
110	74·00	1·80	75·80	74·23	3·47	77·70
135	96·50	3·30	99·80	95·32	7·89	103·21

The limiting pressures on which the section of the Tansa dam was designed were not given in the Paper, but the maximum pressure

[1] *Vide* Fig. 4, Plate 1.

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shown in Fig. 4 was 125·22 lbs. on the square inch at the toe of the outer face, which was equivalent to 8·05 tons on the square foot. The formula was based on a limiting pressure of 8 tons per square foot, with masonry or concrete weighing 140 lbs. to the cubic foot. The Tansa design was based on the supposition that the masonry would weigh 150 lbs. per cubic foot, and its actual weight was about 155·8 lbs. The value of λ in the formula had therefore been taken at 7·25 tons. It would be seen on comparing the figures given in the Table with the profile (Fig. 4) that Gen. Mullins' formulas gave increased breadth just where the calculated lines of pressure approach most closely to, or fell without, the boundaries of the middle third of the section. The outer face of the dam below 85 feet from the top was apparently a straight line tangential to the curve of the upper part of the face at that point, which curve had been struck with a radius of 160 feet. It might perhaps be concluded, both from the figures given by the formula for y and from the calculated pressures at the face, that the profile of this face was not quite as conformable as possible to the conditions to be satisfied; but, on the other hand, for practical purposes it was no doubt sufficiently accurate.

As the water in the reservoir was not likely to fall at any time even to the level of the lowest sluice, which was only 45 feet below the top of the dam at the full height of 135 feet, Fig. 4, the exact position of the line of pressure, reservoir empty, with reference to the middle third of the section, was not of practical importance. That line of pressure, with the water at its lowest level, would certainly fall well within the middle third.

The catchment-area of the Tansa reservoir was of medium size. The maximum rainfall recorded (Appendix A) was 6·79 inches on the 19th July, 1889. It would be necessary to know the rainfall of each of several consecutive hours during exceptionally heavy storms to enable a conclusive judgment to be formed as to what maximum duty the waste-weir might be called upon to perform.

Taking into account the moderating effect of the reservoir, it might be considered that, starting with the water-level with the crest of the waste-weir, the latter would suffice to dispose of a discharge of 1·69 inch from the whole catchment-area in one (the first) hour, and 0·75 inch per hour afterwards. Assuming that experience at the Nagpur reservoir might be taken to indicate that the duration of discharge would be about 1·70 times that of the duration of rainfall, the 1·69 inch would represent a rainfall at the rate of 2·87 inches per hour. The coefficient 0·70 used for

Lieut.-General John Mullins. estimating the depth corresponding to a given discharge passing over the waste-weir, seemed somewhat high, taking into account the width of the crest, which was apparently about 15½ feet. The coefficients of weir-discharges varied with reference to both the depth passing over them and the width of the crests. The information at present available was insufficient for the exact determination of the coefficient proper to any given conditions, but in this case it seemed probable that, for the discharge of 25,000 cubic feet per second, the depth on the crest would approximate to 3 feet instead of 2·5 feet as estimated by the Author. The adjusted coefficient for depth alone was 0·624, and for depth of water and width of crest 0·56. It was easy to ascertain coefficients correctly for very small depths, but the measurement of the large discharges due to considerable depths was difficult to deal with experimentally. Over-estimating the depth required in a case like that in question was an error on the safe side.

The arrangements made for the disposal of surplus water during the building of the dam—a matter of considerable anxiety to engineers in charge of work of this kind—had proved to be entirely satisfactory, and the Author was to be congratulated thereon. A depth of 6 feet of water passing over a dam upwards of 50 feet high was a severe trial, but the form of the outer face was one which greatly reduced shock, and gradually converted nearly vertical to nearly horizontal direction of current. The account of the works for conveying the stored water to Bombay was full of interest, and would be of high value to engineers who might have similar work to perform. The handling and laying of large pipes weighing 3 to 4 tons, and of an aggregate weight of 48,000 tons, required, in a country where suitable appliances were not obtainable, much forethought, and the fact that only one length of pipe failed in the 17½ miles, above Ghat Kopar, showed that the manufacture and testing of the pipes, and the subsequent arrangements for handling and laying them had been efficiently carried out.

Referring to the Baroda Waterworks, he observed that the annual quantity of water to be delivered was 182½ millions of cubic feet. To fill the reservoir, which had a storage capacity of 1,287 millions of cubic feet, a discharge of 15·3 inches or 0·39 of the average annual rainfall from the catchment area would be required, which was more than could be relied on. From the data given in the Paper it appeared that the discharge in the years 1885 to 1889 was in each case less than the capacity of the reservoir. From the diagram, Fig. 3, it would appear that in 1891 also the reservoir did not

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nearly fill, as the water-level in October, at the end of the rainy season, was about 4 feet below the weir-crest. In that year, however, the quantity of water stored was, after allowing for evaporation, sufficient for the supply of Baroda. It was to be hoped that the surplus remaining in the reservoir from good years would make up for the deficiency of other years; otherwise he feared that the reservoir would disappoint the expectations of the designer and of the Government of Baroda. The particulars given of the reservoir embankment showed that the site was unfavourable for the construction of a large impounding-reservoir. About 4,000 feet in length of the dam had to be based on black soil, which was not a satisfactory foundation, as it was liable to considerable changes of condition. It cracked to great depths from the surface in dry weather, and the cracks were sometimes of formidable dimensions. When dry it was very friable, and when moist it was a very soft material. There were, however, many considerable reservoirs which had existed for longer or shorter periods, the embankments of which were composed of and founded upon soil of this description. The manner in which the embankment was formed had shown successful results. At one part, where apparently clay was not found even at a depth of 40 feet, some troublesome and, it might be, dangerous leakage occurred, which had been treated by the precautionary measures adopted, with, it might be hoped, satisfactory results. The waste-weir, the length of which is 800 feet, was probably of ample discharging capacity. Its circumstances were such as to make it difficult to estimate definitely what the discharge would be, but approximately 9,400 cubic feet per second might be taken. The Author had put the discharge at 3,370 cubic feet per second; but that could hardly be correct when the water in the reservoir rose to 212 R. L., or 4 feet above the crest of the weir, which was the intended top-water level. The total cost of the works was Rs.34,40,000, or say £215,000. The outlay, therefore, was £1 15*s.* 10*d.* per head of the population, which was a moderate expenditure if a reliable water-supply had been secured.

The Jeypore Waterworks afforded an interesting example of a well-planned attempt to construct a reservoir under very unfavourable circumstances. The soil on which and with which the impounding embankment had to be thrown up was of so sandy a character that impermeability could not be secured, and the problem was therefore to devise arrangements which would prevent leakage at the place where its consequences might be serious, viz., at the junction of the embankment with the masonry of the

Lieut.-General John Mullins. sluice culvert, and to guard against leakage under and through the embankment, which would lead to the gradual removal of the sub-soil or of the material of the dam.

The reservoir had apparently not filled since its completion in September, 1885, and the embankment had not therefore been subjected to the full head of water; but in that year a depth of 31½ feet of water was retained, or only 9½ feet less than the full depth. Had a water-tight foundation been obtainable, a puddle-wall would have rendered the superstructure impermeable, and then the sand which formed the embankment would have been in most respects a suitable material for the greater part of the dam, though something less liable to disturbance by wind and erosion by rain would still have been desirable as a covering for the slopes, and would have facilitated their protection by the growth of grass.

In this case moderate, or even considerable, leakage did not actually involve loss of water, because such leakage was intercepted by the weir at the pumping-station 750 feet below the reservoir. The reservoir scheme was confessedly one to make the most of a very scanty water-supply. The discharge from the surface of the land was, however, considerably supplemented by springs, with the result that the water in the reservoir was maintained nearly at the level attained at the end of the rainy season for several months; and, altogether, it seemed probable that the water stored would secure a fairly adequate supply to the city during the months when, otherwise, it would have fallen much below requirements.

The arrangements for the disposal of surplus water were not stated in detail, but the channel provided was situated at a safe distance from the dam, and no doubt its requisite capacity of discharge had been carefully considered. Small as was the ordinary discharge from the catchment-area, it might be assumed with certainty that sooner or later one of those exceptional rainstorms, which occasioned so much damage to public works of all kinds, would occur, and then the value of adequate provision for the discharge of flood-water would be appreciated. Probably the means of disposing of 3,600 cubic feet per second, with a depth of 3 feet or 4 feet of water at the crest of the outlet, would be found not an excessive allowance.

Mr. Reynolds. Mr. PLAYFORD REYNOLDS, referring to the Baroda Waterworks, doubted whether the second puddle-trench had any beneficial effect; it was certain that a considerable leak existed under the embankment. He believed that this leak was due to the existence

of porous strata of earth under the embankment. It must be remembered that the puddle-trench in no place reached rock nor even true clay, and a leak of greater or less extent was a certainty. At the same time it was expected that the leak would cease in time, owing to the large amount of silt carried by the river and deposited in the reservoir. He took exception to the measure recommended by Mr. Whiting, who had been called in during Mr. Reynolds' absence in England, if it was correctly described in the Paper. The danger of founding on black soil was that, becoming saturated with water, it might slip or be displaced by the weight of the superincumbent embankment, and the best preventive measure against an accident of that kind was the thorough drainage of the black soil. Mr. Reynolds.

Professor HENRY ROBINSON thought that a Paper on "The Design and Stability of Masonry Dams,"[1] by Mr. W. B. Coventry, might with advantage be studied in connection with the valuable communication of Professor Kreuter. Attention might also be directed to the clear treatment of the same subject in the recently-published book on "Waterworks Engineering," by Messrs. Tudsbery Turner and Brightmore. He had had to advise in regard to the design of masonry dams, and, on more than one occasion, to combat the view that increased strength was gained by augmenting the thickness of the dam beyond the limits ascertained by the recognised methods of determining the profiles. The question of the discharging-capacity of culverts was raised by the Paper on the Tansa works, and he would emphasize the fact that calculations based on the older formulas were erroneous. The results of his investigations of that matter were set forth at length in his work on "Hydraulic Power." Prof. Henry Robinson.

Mr. T. FRAME THOMSON submitted a sketch, *Fig. 23*, showing comparative sections, calculated with the same data of (1) the Tansa dam; (2) an equivalent dam designed by the method described by Professor Kreuter; and (3) an equivalent dam designed by the method given by Messrs. Tudsbery Turner and Brightmore in their recent work on "Waterworks Engineering." As regards economy of material for the particular height of dam in question, viz., 135 feet, Nos. (1) and (3) exceeded No. (2) in area to the extent of 2½ per cent. and 5½ per cent. respectively. The diminished section of No. (2), compared with No. (3), was chiefly due to the equalising portion with an overhanging inner face, introduced by Professor Kreuter, shown hatched in the *Fig.* Mr. Thomson.

[1] Minutes of Proceedings Inst. C.E., vol. lxxxv. p. 281.

Mr. Thomson. The following formula facilitated the plotting of portion No. (3) in Professor Kreuter's method:—

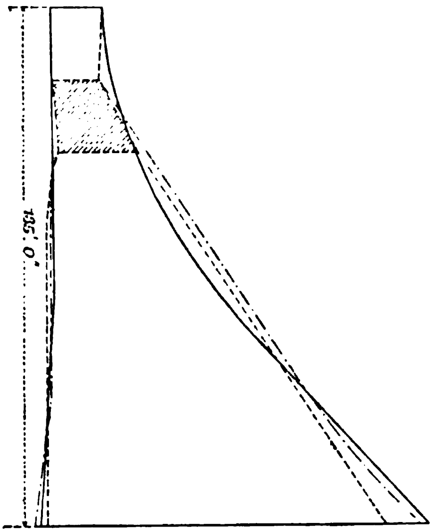
Let a_n be the area of the nth section.

A_n „ total area down to and including the nth section.

g_n „ horizontal distance between the inside toe of the $(n-1)$th section and the centre of gravity of the nth section, plotted with the inner edge vertical under the inner toe of the $(n-1)$th section.

G_n be the distance (one-third of the base) from the inside toe of the nth section at which the line of pressure cuts the base of that section.

Fig. 23.



Scale 1 Inch = 50 feet.

Tansa dam shown thus . . . . ———

Equivalent dam, designed by the method of Kreuter, thus . . . -----

Equivalent dam, designed by Tudsbery Turner and Brightmore, thus —.—.—

p_n be the distance through which the base of the nth section must be moved beyond the vertical through the inner toe of the part above it, in order that the line of pressures (reservoir empty) should pass through it at one-third of its length from the face.

t_n be the thickness of the base of the nth section.

$$\text{Then } p_n = \frac{\frac{t_n}{3} - A_{n-1}G_{n-1} - a_n g_n}{A_{n-1}}.$$

The formula embodied two approximations to the result, and was sufficiently exact for practical use. Mr. Thomson.

Mr. S. Tomlinson thought the following brief description of the works recently completed for the supply of Ahmedabad might be appropriately mentioned in connection with the water-supply of cities in Western India. Ahmedabad was situated 300 miles north of Bombay, near the River Sabarmati, and had a population, according to the census of 1891, of 148,412. The supply was drawn from closed wells sunk in the bed of the River Sabarmati. The four wells were built of brickwork, being 25 feet in diameter, and sunk to depths varying between 8 and 13 feet below summer water-level—their coverings being about 4 feet below the surface of the river-bed. The water was collected from them and conveyed to the jack well under the pumps by 24-inch cast-iron pipes running from the two nearest wells. The pump-well was situated on the banks of the river, and was sunk to a depth of 10 feet. There were two pumps and two engines. The pumps were vertical, with cylinders 28 inches in diameter and plungers 20 inches in diameter. The engines were compound surface-condensing, with 12-inch and 23-inch cylinders of 36-inch stroke. The steam-pressure, generated in Lancashire boilers, varied between 90 lbs. and 100 lbs. per square inch. The delivery-main from the pumps to the elevated tank was of cast-iron, 27 inches in diameter. The suction varied, but might be taken as about 18 feet; the lift was 75 feet. The fuel used was wood, of which about 400 lbs. per hour was consumed at a cost of 4 to 5 pies per 1,000 gallons pumped (192 pies = Rs. 1). There was a high-level tank, which contained 160,000 gallons, supported on a brickwork tower. The tank was of steel, 18 feet deep and 42 feet in diameter, the bottom being 40 feet above the floor-level. It was provided with 27-inch inlet- and outlet-pipes. The outlet-main to the city was about 2½ miles in length. The works were opened in June 1891; the total cost had been Rs. 7,76,800. The scheme was designed to afford a supply of 10 gallons per head per day to the population. The water was supplied for ordinary domestic purposes at a charge of 6¼ per cent. on the assessed value of the premises, with connections, 3⅛ per cent. on those premises without connections, and for trade purposes by meter at 8 annas per 1,000 gallons. The net cost to the Municipality, allowing for interest and sinking fund, was about 2½ annas per 1,000 gallons. Mr. Tomlinson.

On the completion of the Tansa works and the retirement from municipal service of the Author of the Paper, Mr. Tomlinson had taken charge of them, and had much pleasure in stating that the

Mr. Tomlinson. experience of the past year had been of a most satisfactory character. No accident calling for comment had occurred in any portion of the works, and the water had only been shut off once for the annual inspection of the interior of the conduit and for trifling alterations and repairs to sluices, &c. Referring to the rainfall of 1893, the lake overflowed on the 20th June after 26·63 inches of rain had fallen. As the draw-off from the lake was in 1892–93 almost as great as the one line of pipes now laid could carry, that fact showed clearly how small a proportion of the rainfall was utilized.

The highest water-level in 1893 had been 1·75 feet over the waste-weir. The rainfall registered at the dam on five days preceding that record being—

	Inches.
17th June	3·68
18th „	8·53
19th „	7·09
20th „	5·12
21st „	6·22
22nd „	2·24
	32·88

The highest level registered was on the 22nd June at 8 A.M. (406·75.)

The outlet arrangements were simple and efficient, but it was not clear why the sluice-shutters should not have been on the outside of the dam, where the rods would have been exposed and could have been painted or renewed readily.

Observations had been made daily of the temperature of the water in the Tansa lake, and at the upper and lower ends of the siphons.

The particulars might be taken as typical of any 48-inch pipe fully exposed to a tropical sun and discharging about 20,000,000 gallons daily; 8° F. had appeared to be the maximum rise of temperature from the upper to the lower end of a siphon about 11½ miles long. At the north end, 9 A.M., and at the south end, 3 P.M., gave the greatest differences, as might have been expected. It might be noticed that the minimum temperature registered was 73° F. at 6 A.M. on the 1st February, and the maximum temperature was 95° F. at 3 P.M. on the 15th May, 88° F. being the maximum 6 A.M. reading. Those readings accorded with others taken at the Vehar and Tulsi lakes, and at the service-reservoirs in Bombay.

The Tansa water was distributed unfiltered, and the monthly analyses had indicated: Mr. Tomlinson.

		Free Ammonia.	Albu-menoid Ammonia.			Free Ammonia.	Albu-menoid Ammonia.
		Parts per Million.	Parts per Million.			Parts per Million.	Parts per Million.
1892.	8th April	0·06	0·12	1892.	1st Nov.	0·005	0·05
„	30th „	0·05	0·12	„	2nd Dec.	0·01	0·06
„	1st June	0·01	0·06	1893.	5th Jan.	..	0·04
„	4th July	0·05	0·09	„	21st Aug.	0·04	0·05
„	1st Aug.	0·05	0·08	„	3rd Oct.	0·01	0·06
„	1st Sept.	0·005	0·04	„	6th Nov.	0·005	0·10
„	3rd Oct.	0·005	0·04	„	4th Dec.	0·03	0·07

Mr. E. WEGMANN, Jun., considered that the formulas proposed by Professor Kreuter for determining the sections of dams were simple compared with those of Sazilly, Delocre and Pelletreau,[1] and were more general than those given by Rankine[2] for a profile-type bounded by logarithmic curves. He did not think, however, that Professor Kreuter's method satisfied all the practical requirements of the case. Formula No. 1 was simple, though not new, as it was used in 1884 in the calculations for the proposed Quaker Bridge dam. Formula No. 2, which was also very simple, was obtained by making the inner face of the wall overhang the lower portion. As this arrangement saved but a small amount of masonry, and produced some tension in the dam, it was not to be recommended. In all the profile-types which had thus far been proposed, the inner face of the dam was either made vertical near the top, or was slightly battered. If the condition were introduced that the inner face of the wall was to be vertical until a joint was reached where the line of pressure, reservoir empty, was situated one-third of the breadth from the inner face, or where the limiting pressure was reached (as the case might be), Professor Kreuter's formula No. 2 could not be applied. An equation to suit that case could easily be found, but it would contain two unknown quantities (viz., the depth below the top of the dam and the length of the joint), and could, therefore, only be solved by trial. Formulas Nos. 3 and 4 enabled the thickness of the dam to be calculated for that part in which every horizontal joint was trisected by the two extreme lines of pressure (*i.e.*, reservoir full and reservoir Mr. Wegmann, Jun.

[1] See the "Annales des Ponts et Chaussées" for 1853, 1866, 1876, and 1877.

[2] See "Miscellaneous Scientific Papers of Professor Rankine," London, 1881, or the *Engineer*, January 5th, 1872.

Mr. Wegmann, Jun.

empty). The auxiliary expression given in connection with formula No. 4 was not satisfactory as it could not be directly integrated. The problem could be better solved by means of equation 6 of the formula used in designing the Quaker Bridge dam, given hereafter. Professor Kreuter's formula No. 5 would give the dam sufficient thickness to keep the maxima pressures within the fixed limits of safety, but would not determine the minimum area of profile which fulfilled the requirements. An unnecessary amount of masonry in a dam involved, not only a waste of material but also increased pressures in the structure. Whilst formula No. 5 kept the maxima pressures at the up-stream face at the given limit, it did not do so with the maxima pressures at the down-stream face; and consequently made the profile larger than was necessary. By keeping the line of resultant pressures, reservoir empty, on the up-stream limit of the centre third of the profile (a condition for which no necessity was evident), the thickness of the dam was made to increase very rapidly. Professor Kreuter had stated that for a certain depth, formula No. 5 would make the lines of pressure, reservoir full and empty, fall in the same half of the profile. That would lead to an extravagant thickness for a dam of considerable height.

Rankine had recommended that a lower limit of vertical pressure should be assumed for the down-stream than for the up-stream face, in order to compensate for the increased pressures in an inclined direction which occurred when the reservoir was full; and this was probably a safe course. If it were desired to comply with the condition usually adopted by writers on the subject of masonry dams of keeping the maxima pressures at both faces of portion No. 3 exactly at the limit of safety, formula No. 5 could not be applied. With reference to the question of including or omitting the vertical component of the water-pressure in the calculations, he would recommend the latter course for the following reasons: The first writers on the subject of masonry dams—Sazilly and Delocre—assumed limits as low as 4 to 6 tons per square foot for the maxima pressures. With this assumption considerable slope had to be given to the inner face of a dam, and the vertical component of the water-pressure became consequently a factor which could not be omitted in the calculations. Since those writers had designed their profile-types, the tendency in constructing dams of masonry had been steadily towards higher pressures. Rankine adopted a limit of about 8 tons per square foot for the maxima pressures at the outer face and 10 tons at the inner face. With these data, the inner face of a dam became almost vertical for a considerable depth below the top, and the

Mr. Wegmann, Jun.

error (in the direction of safety) resulting from omitting to consider the vertical component of the water-pressure was trifling. For a depth of 160 feet the error in the maxima pressures at the outer face would be only about 5 per cent. with the data assumed by Rankine. By including the vertical component of the water-pressure the calculations became very complicated, and, after all, the actual maxima pressures were not ascertained, and were probably under-estimated, as the usual formulas by which they were calculated did not take into consideration the elasticity of the masonry. Rankine had recommended that the maxima pressures at the down-stream face should be diminished as the slope of the wall increased in order to make allowance for the inclined direction of the stresses at the face of the wall. As in the present state of knowledge, the law of this diminution of pressure was not known, Mr. Wegmann advocated reducing the pressures at the outer face by omitting the vertical component of the water-pressure in the calculation.

The problem of finding the profile of minimum area for a masonry dam was exceedingly complex, as the different portions of the wall were not determined by the same conditions. In the upper courses the positions of the lines of pressure, reservoir full and empty, had to be considered; while in the lower courses the limit of the maxima pressures was the controlling condition. In making calculations for the proposed Quaker Bridge dam (a reservoir wall across the Croton Valley, which was to have a height of about 260 feet from the foundation to the top), he had carefully studied all formulas for determining profiles for dams of which he could find a record, and had endeavoured to apply them to the problem, without much success. The methods of Sazilly, Delocre, and Pelletreau, which were lengthy and complicated, could not be used with high limits of pressure, as they did not confine the lines of pressure to the centre third of the profile. The logarithmic curves proposed by Rankine gave only an approximation to the profile required by the conditions he assumed. Krantz, Crugnola, and Harlacher had published good profile-types, but had given no formulas to vary them for different data. Sir Guilford Molesworth had given an empirical formula for determining the profile of a dam, and Mr. W. B. Coventry had proposed a method [1] consisting partly of trial calculations and partly of the use of equations.

All the formulas proposed for determining the profile of a dam

[1] Minutes of Proceedings Inst. C.E., vol. lxxxv. p. 281.

Mr. Wegmann, Jun. (including those of Professor Kreuter) involved, more or less, trial calculations. With the complicated conditions of the problem it was not likely that anyone would succeed in finding exact general formulas for determining the profile of minimum area. While it was difficult to find at once a practical profile fulfilling the given conditions, it was comparatively easy to determine the exact thickness of the dam, at regular intervals, commencing at the top. If the intervals were taken sufficiently small (say, 10 feet), and the vertical component of the water-pressure were omitted, a correct theoretical profile of a dam could be calculated by the equations which were used for the Quaker Bridge dam, fully set forth in a work written by him a few years ago.[1] He had determined the first profiles for the Quaker Bridge Dam, to fulfil the conditions laid down by the Chief Engineer, by a simple but rather laborious system of trial calculation, consisting in commencing at the top of the dam, and finding the required thickness at regular intervals by trial, verifying the conditions by taking moments about a vertical axis. He had subsequently improved this rather primitive system by finding correct equations. The profile found by those equations would have polygonal faces, but by taking the depths of the several layers considered sufficiently small the profile of minimum area could be approached as closely as desired.

Having found the profile which contained practically the minimum area which fulfilled the given conditions, it only remained to simplify its outlines by substituting a few straight lines or curves for the angles of the polygonal faces, in order to obtain a practical design. Small changes in the outlines for this purpose had no appreciable effect. The profile obtained in this manner might not fulfil the given conditions exactly, but it would certainly do so practically; and as the distribution of pressures in masonry was unknown, it seemed to be a useless mathematical refinement to insist on greater accuracy than that which resulted from the method referred to. The reservoir wall, known now as the New Croton dam, was being constructed at a place $1\frac{1}{8}$ mile further up-stream than its original location. Its maximum height above the foundation would be about 240 feet. The storage-reservoir for New York, which it would form in the Croton Valley, would contain about 30,000,000,000 United States gallons.

Mr. Whiting. Mr. J. E. Whiting, referring to the form of the masonry conduits and lined tunnels of the Tansa works, thought it manifest that the

[1] "The Design and Construction of Masonry Dams." New York, 1888.

lower corners should have been rounded off, the extra cost of a slightly modified section would not have been great, and its advantage as regards cleanliness would have been considerable. Nothing had been stated in this Paper as to the quality of the water from Tansa; that it was better than the water received from Vehar and Tulsi appeared from the following analyses: Mr. Whiting.

Analysis taken 6th November, 1893, of water drawn from the Tansa main at the outskirts of the city: free ammonia, 0·005 part per million; albumenoid, 0·10 part per million. Tulsi water gave free ammonia from 0·03 to 0·04 part per million; albumenoid, 0·12 to 0·14 part per million. Vehar water gave free ammonia 0·03 to 0·04 part per million; albumenoid, 0·12 to 0·15 part per million. An analysis of Tansa water taken from the main on 8th June, 1893, showed, total solids, 9·10 grains per gallon; chlorine, 0·98 grain per gallon; free ammonia, 0·03 part per million; albumenoid, 0·09 part per million.

When considering the splendid work described, it seemed a pity that either distinct provision had not been made for the utilization of the whole storage hereafter, or that the dam had not been designed of less thickness. It appeared that the lined tunnels and conduits would carry about double what could pass through the one cast-iron main already laid. The aqueducts across the Bassein creek and elsewhere were constructed to carry a second pipe, and that Bombay would ere long require the second main was probable. But, if the double storage should be required, how would the extra water be got out of the reservoir? The present outlets, would apparently not permit that; fresh tunnels could perhaps be formed, and the conduits be doubled in width. Probably new cylinders would have to be sunk for extra girders across the creeks; or could the present iron-work be strengthened and the new pipes be carried on the top of the present ones? All those matters might be arranged, but new outlets through the dam would be troublesome to make. As regarded the bursting of some of the pipes, it was to be regretted that the Author had not given his reasons for inclining to the opinion that the want of air-valves in that portion of the main was the cause of failure. Air in a pipe could not of itself increase the pressure; on the other hand, if any hydraulic ram occurred in a pipe, the presence of air would modify that and reduce its bad effects.

Mr. Clerke. Mr. CLERKE, in reply to the correspondence, observed that the Tansa pipe-joint, though closely following that published by Mr. Fanning some years ago, was not identical with it. The dimension *m n*, on the drawing of the Tansa pipe, exceeded the cor-

Mr. Clerke. responding dimension of Mr. Fanning's drawing by $\frac{1}{3}$ inch, giving greater substance in the shoulder of the socket and adding 80 lbs. to the weight of the pipe. The form and dimensions of the spigot of the Tansa pipe differed from those of Mr. Fanning's. With reference to Lieut.-Gen. Mullins' remarks on the cost of storage at Tansa, he must observe that the necessity for drawing off at a high level to command Bombay left a much greater quantity of water unutilized than would be the case in a similar work constructed for irrigation purposes. In the latter case, the cost of the storage at Tansa would be about Rs.705 per million cubic feet. The information given by Mr. S. Tomlinson in reference to the maximum flood of 1893 and the rainfall immediately preceding it, emphasized Mr. Clerke's remarks in his reply to the discussion, in which he had pointed out that the rainfall at the dam site could not be regarded as an index of the general rainfall on the catchment-area. He had to express his gratification at the interesting and able remarks of both a practical and theoretical nature on the general subject of masonry dams, which the discussion and correspondence on the group of papers had elicited from the engineers who had taken part therein.

28 November, 1893.

J. WOLFE BARRY, Vice-President, in the Chair.

The discussion upon the Papers on Impounding Reservoirs in India and the Design of Masonry Dams occupied the evening.

5 December, 1893.

Sir ROBERT RAWLINSON, K.C.B., Vice-President, in the Chair.

It was announced that the following Associate Members had been transferred to the class of

Members.

Harold William Abernethy.
James Herbert Bartlett.
Arthur Henry Birkinshaw.
John Henry Briggs.
Arthur Edward Brown.
Matthew Joseph Butler.
Albert Havelock Case, Wh.Sc.
Maurice Fitzmaurice, B.E.
Everard Hesketh.
Harrison Hodgson.
William Field How, Wh.Sc.
Robert Edward Jones.
Matthie Charles Mackinnon.
John Monthermer Montague, M.A.
George Philips Mulock.
Thomas Ormiston Paterson.
Augustus Tichborne Pentland, M.A.
Edward Bellingham Price, B.A.
John Henry Horace Wentworth Rhodes.
Henry Smith.
William John Patrickson Storey.
George William Sutcliffe, Wh.Sc.
John Edward Worth.

Also that the following Candidates had been admitted as

Students.

Robert Arnolds Becher.
Edgar Lionel Benjamin.
Thomas Rolph Bennett.
Wilfred Gilbert Bower.
Oliver James Collis.
Geoffrey Scott Dalgleish, B.Sc.
William Davidson.
Frederick William Daniel Davis, B.Sc.
John Frederic Day.
Henry Denton.
Audley Mervyn Duke.
Walter Inglis Ferrar.
William James Firth.
Sydney Harold Garnett.
Frank St. John Gebbie, B.Sc.
Nicholas George Gedye.
George Archibald Graham.
Allan Dickson Grigg.
Thomas Ernest Hack.
Francis Ortner Harrison.
Louis Edward Heberden.

Students—continued.

George Herbert Heelas.
Thomas Hewson, Jun.
Francis Eliot Howard.
John Lett Sealy Jeffares, B.Sc.
Francis Edgar Kanthack.
Harvis Jürgen Karl Kühl.
Alexander Lade.
William Henry Ledger, B.C.E.
Francis Samuel L'Estrange.
Matthew Loam.
Owen David Lucas.
Alexander Lawrence McKenzie.
Roland Marsland.
James Edward Moberly.
Arthur Moseley.
Edward Hugh Dyneley Nicolls.
Herbert Nuttall.
Frank Parkin.
William Samuel Payne.
William Edwin Pincombe.
William Ross Rae.
Hugh Percival Raikes.
Rupert Peel Richards.
Heaton Forbes Robinson.
William Salmond.
Charles Benjamin Saner.
William Peter Sangster.
John Shaw.
Ernest Martin Sheppard.
William Croasdill Shettle.
Albert Edward Harvey Sonneborn.
Walter Charles Stanton.
Frederick Marshall Stuart.
William Tom Wood Sussman.
Edward William Swinstead.
Joseph Charles Sykes.
Alfred John Terry.
Logan Millar Tod.
Pierce Joseph Tucker.
Edward Percy Unwin.
Samuel Joseph Lee Vincent.
Philip Ridsdale Warren.
Charles Elton Wearing.
Thomas Henry Wells, B.A.
James Whyte.
Llewellyn Williams.
Arthur Worthington.
Henry Hodgson Wright.

The following Candidates were balloted for and duly elected as

Members.

William Hubert Burr.
Edward Augustus Hackett, M.E.
James Thomas Jervis.
Walter Mansell.
William Ripper.
Frank Robinow.
Edward Clapp Shankland.

Associate Members.

Thomas Aitken.
Marcus Allen.
John Logie Allison.
Charles Davidson Barker, Stud. Inst. C.E.
Thomas Gibson Barlow-Massicks.
George Baxter.
Archibald Græme Bell.
Anderson Webb Blew.
Hubert Francis Toriano Bode.
James William Bradley, Stud. Inst. C.E.
Victor Edgar de Birchin de Broë.
John Walter Brown, Stud. Inst. C.E.
George Elliot Browning.
George Frank Burn, Stud. Inst. C.E.
John Mindoro Cameron.
David Carnegie, Stud. Inst. C.E.
George Chalmers.
John Henry Chaloner Chute, Stud. Inst. C.E.
James Laughlin Clark.
Stamford Vair Clirehugh, Stud. Inst. C.E.
Wilberforce Cobbett.
John Pearson Copland, Stud. Inst. C.E.
William Corin, Stud. Inst. C.E.
Arthur Chamier Davis.

Associate Members—continued.

Frederic John Dawson, Stud. Inst. C.E.
Nelson Frederick Dennis.
Harry Arthur Dix.
Hawtrey Marks Drummond, Stud. Inst. C.E.
Richard John Durley, B.Sc., Wh.Sc., Stud. Inst. C.E.
Edward Percy Fairbairn.
Richard Fennelly.
Francis Douglas Fox, B.A., Stud. Inst. C.E.
Alfredo José Nabuco de Araujo Freitas, B.Sc.
Astley Paston Friend.
Henry Hannay Fuhr, Stud. Inst. C.E.
George James Furness, Stud. Inst. C.E.
William Morris Gale, Stud. Inst. C.E.
William Gilbert, Wh.Sc.
Herbert Charles Gostling, Stud. Inst. C.E.
Norman Greenshields.
Edwin Walter Greenwell, Stud. Inst. C.E.
Claude Greenwood.
Albert Edwin Griffin.
André Pierre Griffiths, B.Sc., Wh.Sc., Assoc. R.S.M.
Harry Hancock.
William Hardie, Jun.
Louis Harper.
Tom Harold Hawkins.
Cecil Edward Hawley.
John Heathcott, M.Sc., Stud. Inst. C.E.
Arthur James Henderson.
Robert Henderson.
William George Holford.
Arthur Charles James, Stud. Inst. C.E.
Henry Holmes Jellett, B.A., B.E.
John Henry Johns.
Joseph Malcolm Kerr.
Harold Werner Kolle.
Albert Francis de Labastide.
Ferdinand Joseph de Jacques de Labastide.
Ernest Matthew Lacey.
Gilbert Laing-Meason.
Fitz-Gerald Gage Lambert.
William Henry Fitzroy Landon.
Frank Laurens, Stud. Inst. C.E.
Charles William Ruffle Lawson.
Samuel LeCocq.
John Thorpe Lewis, Stud. Inst. C.E.
Arthur Edward Lightbody.
Herbert Samuel Heelas Macaulay.
Henry Marten.
George Henry Martin.
James Newsome Matthews, Stud. Inst. C.E.
Walter May (*Engr.* R.N.).
Charles Walter Mills.
John William Moncur.
Bishambar Nath (*Rai Saheb*).
Frank Edward Berra Nathan, Stud Inst. C.E.
Arthur Oliver Nicolls, Stud. Inst. C.E.
Alfred Mark Oliver.
James Harvey Williamson Park, B.Sc.
Philip George William Parkman, Stud. Inst. C.E.
Hubert Pooley, Stud. Inst. C.E.
Leopoldo de Abreu Prado.
Ernest Prebble, Stud. Inst. C.E.
Frederick Charles Preston, Stud. Inst. C.E.
Acheson Lyle Rathbone, Stud. Inst. C.E.
Robert Newby Hartley Reid, Stud. Inst. C.E.
Alexander Gordon Robertson, Stud. Inst. C.E.
John Rogers.
Edward Frank Sargeant.
Emile Schwoerer.
William Henry Severn.
Philip Alan Sharman.
Edward Stockley Sinnott, Stud. Inst. C.E.
Stirling Smeaton, B.A.
Harold Duke Smith, Stud. Inst. C.E.
Harry Colvin Smith, Stud. Inst. C.E.

Associate Members—continued.

HARRY READ STOCKMAN.
ATHELSTAN PHILIP JOSEPH STOURTON, Stud. Inst. C.E.
CHARLES LEOPOLD STRAPP, Stud. Inst. C.E.
JOHN TATE.
ALEXANDER JAMES WALLIS TAYLER.
STANHOPE EVELYN THORNTON, Stud. Inst. C.E.
RICHARD FENWICK THORP, Stud. Inst. C.E.
WILLIAM ELPHINSTONE UNDERWOOD, Stud. Inst. C.E.
PRESCOTT UPTON.
ARTHUR EVAN VIGERS, Stud. Inst. C.E.
CHARLES JAMES POWER WALLER, Stud. Inst. C.E.
FREDERICK MACKMAN WATSON.
STEPHEN LLOYD WHETHAM, Stud. Inst. C.E.
GEORGE REGINALD WHITAKER.
CHARLES ARTHUR WHITE.
ROBERT HENRY WHITE.
JOHN WHITELAW, Stud. Inst. C.E.
EDWARD TURNER WHITELOW.
WILLIAM HUMPHREYS WILLIAMS.
JOHN ROBERT ROBINSON WILSON.
NORMAN MCLEOD RAMSAY WILSON, Stud. Inst. C.E.
WILLIAM RICHARD WOOD, Stud. Inst. C.E.
RICHARD GEORGE HANSFORD WORTH, Stud. Inst. C.E.
JAMES YOUNG, Stud. Inst. C.E.

Associates.

EDWARD MACFARLANE.
JAMES STEWART (*Capt.* R.E.).

The discussion upon the Papers on Impounding Reservoirs in India and the Design of Masonry Dams was resumed and concluded.

12 December, 1893.

SIR BENJAMIN BAKER, K.C.M.G., Vice-President, in the Chair.

(*Paper No. 2745.*)

"Cask-Making Machinery."

By Lewis Henry Ransome, Assoc. M. Inst. C.E.

The growing demand for casks and barrels of all kinds, has caused engineers and others for some time past to devote much attention to designing machines for dealing with this class of work; though, owing to certain inherent difficulties, for some time little progress was made.

The first patent for machinery to be used in the manufacture of casks was obtained so long ago as 1825, and, though numerous inventions (a large percentage of which are worthless for practical purposes) have since then been brought out, it is only comparatively recently that such machinery has come into general use.

The chief difficulties to be overcome were: 1. The different sizes and varying shapes of barrels; 2. The great diversity of material employed; 3. The difficulty of working wood for this purpose by machinery without undue waste of material; 4. The prejudice of skilled coopers, who have always dreaded the introduction of machinery, which has been regarded as likely to deprive them of their livelihood. It has now been proved beyond question that every operation connected with the manufacture of casks and barrels can be far more quickly and economically carried out by machinery than by hand; and the samples (exhibited) of different types of casks and barrels employed in commerce, all of which have been made by machinery, may bear out this assertion.

Casks may be divided, roughly speaking, into three classes, viz., slack casks, for holding substances such as flour and cement; semi-tight casks, for gunpowder, butter, &c.; and tight casks, to contain wine, spirits, beer, or other liquids. The different forms of machines used for these varieties are very numerous. It is proposed, therefore, to confine this Paper to the description of the manufacture of one type of slack barrel, and one of a tight cask.

In dealing with the manufacture of both types, it must be understood that the description applies only to casks made in the United Kingdom, as, in other countries, certain modifications in manufacture are necessary to suit the local exigencies of the trade. It may also be added that, while many machines in use for accomplishing the same purposes are necessarily omitted, the present selection is limited to those which experience has shown to give the best results, to the design of which English, German and American inventors have contributed.

Perhaps the best example of slack barrel is afforded by that used for cement, of which many hundreds of thousands are made every month. Cement-barrels are usually of fir, the staves being ½ inch thick, and the barrel is 28½ inches high by 16½ inches in diameter at the head, with a slight bilge. The staves are imported sawn to the requisite length and thickness, and vary in width between 2½ inches and 5 inches. The first operation is to joint them, that is to say, to plane them on both edges, making the two ends narrower than the middle, so that when drawn into shape, the barrel shall be of the proper form, the edge of each stave bearing against the next one throughout its entire length. In this style of barrel it is sufficient for the joints of the staves to be rectangular, as when trussing thin staves of soft wood, the pressure coming chiefly on their inner edges, the centres are forced outwards, and the wood takes a convex form on the outside. The machine which joints the staves is illustrated in Fig. 1, Plate 5, and consists of a bench with a planed table, in the centre of which is a revolving cutter-block, hinged to allow of its rising and falling through a gap in the centre of the table. At one end of the cutter-block is a chilled cast-iron roller, which rests on a wrought-iron template, hollowed out to the curve to which the edges of the staves are to be planed. This template is attached to a frame which travels automatically backwards and forwards over the cutter-block, and the roller is kept down on to the template by a weight attached to the cutter-block carriage. A number of staves are placed on edge on the table, and are held securely in the travelling frame by a cramp worked by a hand-wheel and screw. When the machine is set in motion, the frame passes over the cutter-block, which falls and rises as the template moves along, and so dresses the edges to the required curve. The staves are then taken out, and the carriage is run back with a quick return motion, after which they are turned over upon their other edges, and are passed over the cutters a second time. It will be seen that by means of this machine, staves of various widths can be jointed; and, as the cutters can

be set back on the block so as only just to project through the table when the centre of the template is opposite the centre of the gap, there is no waste. As, however, the wider staves require more curve on the edges than those which are narrower, it is necessary to sort them into approximately equal widths, and use a different template to each. Mr. E. E. Whitehead, of Wouldham, has recently invented an ingenious revolving template, Fig. 2, with four wings, each one being hollowed out to a different curve, by turning which the different curves required for the various widths of staves can be brought into use without the trouble and delay of changing the templates. This machine will joint sufficient staves for seventy barrels in an hour; a skilled cooper can joint about ten sets in that time.

After being jointed, the staves are heated in order to allow of the barrel being trussed up without breaking. For this purpose a cast-iron table is used, heated by a furnace below. The staves are laid in sets upon this hot plate until sufficiently heated, after which they are ready for trussing. This operation, in which the staves are forced into hoops, in order to bring the barrel into its proper shape, is most important, as, unless it is properly performed, the barrel will not be strong enough to stand the wear to which it is likely to be subjected. The trussing-machine consists of a strong cast-iron cone-shaped bell supported by two turned steel uprights, and hinged in the centre so that it can be opened, as shown in Fig. 3, Plate 5. The inside of this cone is turned to the shape of the outside of the barrel, and two circular grooves are cut in it to receive the iron truss-hoops. The uprights which carry the cone also form guides for a table which is worked by hydraulic pressure, and has a rising and falling motion. The barrel is "raised" in the bell in the following manner: Two truss-hoops (one for the head and one for the quarter) are inserted in the grooves, and the bell is closed. A bilge truss-hoop is then laid in the centre of the table. A boy arranges the staves in a circle with their lower ends inside the hoop on the table, while the upper ends rest against the inside of the cone. When the hoop is filled with staves, the table is caused to rise by turning a handle which opens a valve from the accumulator, and the staves are forced into the cone. As the top ends close together, the bottom ends spread out against the bilge-hoop on the table. The cone is then opened, and the barrel is taken out with the three hoops on it, the "head" and "quarter" hoops being in their proper positions, while the "bilge"-hoop is still at the bottom of the barrel. Two more hoops are then inserted in the cone, the table is lowered, and

the barrel, being reversed upon it, the operation is repeated. In the latter case, as the barrel rises, the bilge-hoop is caught by a shoulder turned on the inside of the bell in such a position as to bring the hoop to the middle of the barrel. The whole operation of setting-up the staves and trussing the barrel is performed with great rapidity; and a boy, accustomed to the work, can both set-up and complete the trussing within a minute—an operation which, if performed by hand, could not be accomplished by a skilled workman in less than ten minutes.

In cement barrels, the head is held between two wooden hoops nailed to the inside, but it is necessary to bevil the inner ends of the staves so that the head may be easily inserted. This operation is performed by the chiming machine, illustrated in Fig. 4. In working this machine, the barrel, after being trussed, is laid with the truss-hoops still on in the centre of the bed. Two cast-iron sliding chucks are brought together by a lever in front of the machine, and, embracing the ends of the barrel securely, hold it in position. After this, the cutter-blocks, which are driven from overhead by wooden drums, are brought forward simultaneously inside the ends of the barrel by means of another lever at one end of the machine, and so bevil the two ends. This finishes the manufacture of the body. A machine of this description will chime two hundred and fifty barrels per hour.

Turning the head is a very simple process. There is no bevil required on the edge, and it is made up of three or four narrow pieces of wood which are not even jointed or dowelled together. The machine generally employed for this purpose is illustrated in Fig. 5, and turns two heads at a time. The loose pieces which are to form the head are cramped side by side between two cast-iron circular plates, and one head is laid on the top of the other, the grain of the wood of one of them being placed at right-angles to the grain of the other, in order to facilitate the cutting. The lower plate is driven at a high speed, while the upper, which is loose, is furnished with projecting steel set-screws with sharpened points. By pressing down a lever shown on the drawing, the belt is brought into action and the machine is set in motion. A parting-tool, working in a vertical slide, cuts the heads to the required diameter. On releasing the lever, the belt that drives the machine is thrown off automatically, and the heads are released. It should be mentioned that the upper cramping-plate is furnished with two wrought-iron strips and springs, which, when the top chuck is raised, press the strips forward, thus pushing the heads off the spikes, and leaving them free to be

removed. The output of this machine is seventy-five pairs of heads per hour.

Both wooden and wrought-iron hoops are used for cement barrels. The former are made by hand, generally of hazel, chestnut, elm, or any similar wood which may grow in the district. The iron hoops are cut to length by a machine which simultaneously shears off the ends and punches the holes for the rivets. This machine makes sixty strokes a minute, and a boy, after a little practice, is able to cut off and punch a hoop at every stroke. After being punched and cut to length, the hoops are passed between rollers of hardened steel. Both these rollers are driven, and the top one is furnished with spiral grooves. By weighting one end of the top roller-spindle more than the other, one edge of the hoop is stretched, so that when bent round it assumes a conical form to fit the shape of the barrel. An adjustable guide at the back of the machine bends each hoop to the required circumference as it is passed through the rollers. After being rolled on this machine, the ends are brought together, a rivet is inserted through the holes, and a cast-iron plunger, carrying a steel riveting-tool, descending upon an anvil, rivets them together. The remainder of the work is done by hand, the head being placed on the hoop nailed inside the barrel and a second hoop fixed above it. The barrel is then bound with the wooden and iron hoops and is ready for use. The principal centre of the cement industry in England is on the banks of the Thames and Medway, where there are no less than fourteen factories, which manufacture their barrels by machinery. In addition to the cooperages in that district, there are others in various parts of the kingdom, the total output of which is not far short of ten millions of barrels per annum.

The manufacture of tight casks is a much more complicated process than the foregoing, and hardly resembles that of slack barrels in any particular. Owing to the necessity for better finish and greater accuracy, a tight cask cannot be made with anything like the rapidity attained in the case of slack barrels, and this is especially the case with beer casks, the staves of which usually range from 1 inch to $1\frac{3}{4}$ inch in thickness. The oak staves from which brewers' casks in this country are made, are mostly imported from the Baltic, the chief markets being Memel and Danzig. Before being shipped, they are roughly dressed to a rectangular section, and vary between 15 inches and 72 inches in length, between $1\frac{1}{2}$ inch and 3 inches in thickness, and between $2\frac{1}{2}$ inches and 6 inches in width. They

are converted by the cooper, with an ordinary band-saw, into three classes, according to the requirements of his customers, viz., "straight cut," "doublets," and "tonguers," as shown by Figs. 6, Plate 5.

There is a great difference of opinion as to the best method of jointing staves for tight work. When jointed by hand, the edges are first roughly dressed with an adze, and finished with a plane, the cooper relying upon his eye and manual skill to give the right amount of curve and bevil, upon which the excellence of the joint entirely depends. It may here be mentioned that the staves should be bevilled so that, when the cask is trussed up, the pressure will come chiefly on the inside edges; as, if the joints should be open on the inside, the liquor would run down between them, and the cask would leak at the ends; besides which, joints open on the inside allow impurities to collect between them and render the proper cleansing of the cask impossible. When machinery was first introduced for this work, it was found that the joint made by a circular saw properly sharpened and set was absolutely water-tight. As, however, planed joints are sometimes preferred, it will be well to describe machines for making both kinds.

The saw stave-jointer consists of a bench carrying a saw about 18 inches in diameter, as shown in Fig. 7. A long steel template-bar, bent approximately to the required curve, is attached to the top of the table. This template is fixed at the centre just opposite the cutting edge of the saw, but is adjustable laterally by a series of screws projecting at intervals through the side of the table, by means of which it may be bent to suit staves of different widths. The stave to be jointed is placed on a carriage sliding upon the template and is cramped securely endwise by means of a hand-lever, which brings forward a toothed dog and is fed past the saw by hand. The attendant takes care that the edge of the stave does not project too far, in order that the saw may only remove as much wood from the centre as is necessary to make a perfect joint. The stave is then turned end for end, and the other edge is trimmed in the same manner. A graduated scale is cut at each end of the cramp, to ensure the two ends being made of equal width. The carriage which holds the stave can be set to any angle with the saw to give the requisite bevil to the edge. This machine will joint one hundred and twenty hogshead staves per hour. A skilled cooper can joint about twenty in the same time.

There are various machines for producing a planed joint. The one in most general use is illustrated in Fig. 8. In it two or more staves are placed one above the other on a sliding

carriage. Two rows of pins, arranged vertically opposite the edges of the staves, are brought against them by a lever. Each pair of pins is separately weighted, so that they adapt themselves to staves of different widths, and thus push them against a movable fence on the further side of the machine. When brought into position, the staves are securely cramped by a hand-wheel and screw on the top of the machine. When the foot-lever shown in the Fig. is depressed, the fence at the back is withdrawn clear of the staves, and the carriage, worked by a rack-and-pinion, is set in motion. The cutters, shown in a separate view, are attached to a vertical spindle at the back of the machine, which has a rising and falling motion imparted to it by a cam. As the staves pass along, the cutters rise, and being shaped to the bevil required, they remove less and less wood until the centre of the staves is reached; after which they gradually fall again until the staves have passed, thus making them narrower at the ends, and giving them the proper form. As a charge of staves can be jointed during both the forward and return motions of the carriage, the output of this machine exceeds that of the saw-jointer.

After being jointed, the staves must be backed and hollowed, that is to say, the back of each stave must be planed convex, and dressed to a curve which is an arc of the circle of the outside circumference of the cask; while the inside must be hollowed out to allow of its being more readily bent, the hollow being deepest at the centre, and gradually dying out as it approaches within 2 inches or 3 inches of the ends, which are thus left the full thickness for strength. The backing and hollowing of these staves, which are more or less crooked, require a special machine (Fig. 9), as the cutters which perform the operation must not only follow the curves and twists of the staves without wasting the wood, but must, at the same time, plane both edges to the same thickness. In this machine the stave is carried past the cutters by means of two parallel chains driven by tumblers revolving at one end. These chains are united at intervals by dogs, hinged to allow of their rising and falling. It is important that the stave before arriving at the cutters should be centred, so that it passes in a straight line between them; as the cutters, being curved, would not otherwise plane it to an equal thickness on both edges. For this purpose, two pairs of centering-pins are provided, each pair acting upon one end of the stave. By pressing down the foot-lever, they are opened out, the stave to be dressed is placed between them, and, the pressure on the lever being removed, the pins are brought together simultaneously by weights, thus placing the

stave in position in the centre of the table. They are also so arranged that as the dogs pass along they are depressed below the surface of the table, but rise again as each dog passes, ready for the next stave.

The cutter-block to which the hollowing-knives are fixed is placed immediately above the one which planes the backs. On either side of these cutter-blocks, the table over which the staves pass is raised slightly for a length of about 6 inches. This leaves the stave free whilst being operated upon to rock about on the table, while the pressure at the front and back of the cutter-blocks securely holds the portion which is being cut, but does not interfere with the stave adapting itself to the cutters as it passes along, and thus its original form is maintained. The upper cutter-block, it should be mentioned, has a rising and falling motion on a vertical slide, and is actuated by levers worked by a cam keyed on to the same shaft as the tumblers which drive the feed-chains. It is arranged to lift the cutters clear of the stave as it enters, and to allow them to descend gradually until the centre of the stave is reached, after which they rise again, leaving both ends alike. Of course, for different lengths of staves, exchange cams must be used, and the dogs which carry the staves must be spaced to suit. This machine will back and hollow two hundred to three hundred staves per hour, according to size. A cooper can finish from twenty to thirty by hand in that time.

After being backed and hollowed, the next operation is to "raise" the cask. The apparatus used in this process consists (Fig. 10) of a round plate with a circular flange cast on it, which is slightly smaller in diameter than the inside diameter of the end of the cask. Outside this a loose hoop is placed, and, a few inches above the latter, a somewhat larger hoop is held by wrought-iron supports. The staves are placed side by side on end inside these hoops, and the last stave which goes in must be of a "driving" fit. This apparatus, though simple, is important, as it insures the same quantity of wood being used in each cask, and consequently each one has the same capacity. Moreover, by its use, a boy can raise a cask—an operation which would otherwise need to be performed by a skilled cooper.

After the cask has been raised, it is placed in a chamber into which steam under pressure is admitted. Here it is left until it becomes soft and pliable. It is next taken to the "windlassing" machine (Fig. 11), which gathers up the open ends for "trussing." This machine consists of a wooden table, at one end of which is a revolving drum. On this drum is wound a chain, to the end of which is fixed a double hook. After the cask is placed in the

position shown in the drawing, a wire rope, both ends of which are attached to the double hook at the end of the chain, is passed over the staves so as to embrace their ends, and the machine is set in motion by means of a foot-lever which actuates a friction-clutch working a worm and worm-wheel. The drum on which the rope is wound being conical, gathers the cask up quickly at first, and more slowly as the staves are brought closer together and require greater power. When drawn up, a hoop is dropped on to keep them together, and the machine is thrown out of gear. After being windlassed, the cask is taken to the trussing machine (Fig. 12). The chief part of this machine is below the floor, the only parts above it being the five dogs shown on the drawing, and the lever that sets the machine in action.

The dogs referred to are each provided with a clip of hardened steel which lays hold of the truss-hoop, and are moved up and down by a screw driven by bevil-wheels. They are connected together by means of a wrought-iron ring, to which a series of cranks is attached in such a way that, when one dog is moved in or out, the same motion is imparted to all. The operator, having slipped the truss-hoops over one end of the cask, places it on the floor in the centre of the machine as shown. The dogs, actuated by springs, being released, close in against the sides of the cask above the truss-hoop, and, descending slowly, draw it down until the joints of the staves are close, when they are caused to rise again with a quick motion, and the operation is repeated until all the hoops are in position. The bilge-hoop should be first pulled down, as it supports the cask at the middle and keeps it in shape while the other hoops are being drawn on. The work accomplished by this machine is equal to that of six men, as it will truss twenty casks in an hour.

The next machine (Fig. 13) to which the cask is taken, is, perhaps, the most valuable of all in saving labour, as it performs the various operations of "chiming," "crozing" and "howelling" both ends of the cask simultaneously, besides trimming off the ends of the staves to an equal length. The chime, as previously mentioned, is the bevil on the ends of the staves; the croze is the groove into which the head is fitted; and howelling is the operation of adzing the staves to make the croze of uniform depth throughout. Fig. 14 represents a section of the end of a stave which shows the various parts referred to. As in the case of the chiming machine for slack barrels, the cask is clamped securely between chucks, which, in this case, are caused to revolve slowly. The cutters which perform the various operations are all mounted on

one cutter-block, and both ends of the cask are worked simultaneously by two cutter-blocks, one at each end. These blocks are mounted on hinged carriages, which are brought into action together by means of a hand-wheel and screw in front of the machine, while the rotation of the cask is accomplished by means of mechanism thrown into gear by the foot-lever shown in the Fig. As the cask is finished in one revolution, the machine completes its work with great rapidity, turning out from thirty to forty casks per hour, according to size—an output which is equal to that of nearly twenty hand-coopers.

As the various cutters on this machine have to be set with the greatest accuracy, and, as may be readily understood, it is a matter of considerable difficulty to adjust them properly in their places on the spindles, a simple apparatus has been devised for setting them on the blocks before fixing them on the spindles of the machine. This apparatus consists of a template cut to the exact shape of the ends of the staves, and a horizontal spindle on which the cutter-block is fixed. As the workman can handle the block in this position with much greater ease, and can also set his cutters with absolute accuracy, their adjustment is effected in at least one-fourth part of the time in which it can be done on the machine. After being chimed and crozed, the cask is taken to a "cleaning-off" machine, in which it is cramped between two revolving plates, and the operator, by means of a plane suspended from a bar overhead, cleans off the outside.

The head of a beer-cask is made up of three or four pieces of wood, which are planed on one face and jointed and dowelled together. The planing is done on an ordinary hand-feed planer. After the face side has been planed, the edges are jointed on the same machine. For this purpose a fence is provided, and the wood is passed over the cutters, the planed face being pressed against the fence to insure the joint being rectangular. The joints of the heads of tight casks are always made slightly hollow, as it is found that, when exposed to the weather, the tendency is for the joints to open at the ends. The dowel-holes are bored by a small auger fitted into the end of the planing-spindle, the heads being laid on a table which can be adjusted to suit the various thicknesses, and being fed up to the auger by hand. The dowels are wooden pegs, $\frac{3}{8}$ inch diameter, pointed at each end. After the pieces have been jointed and dowelled, the pegs are driven into the holes and put together by hand.

The finished head must be slightly oval, as, when placed in the cask and hooped up, the external pressure brought to bear on it is

considerable; and, as the wood is of course much more easily compressed, and also has a greater tendency to shrink lengthwise than endwise of the grain, it is necessary to make the head from about $\frac{1}{8}$ inch to $\frac{1}{2}$ inch oval, according to size, to allow for this. The ovalling and bevilling is effected in one operation—the best machine for this purpose being illustrated by Fig. 15. The head is cramped between two cast-iron plates, the lower of which is driven; and the machine is so arranged that, when the head is completed at one revolution, it stops automatically. Two cutter-blocks are employed, each carrying three pairs of knives, one of which cuts the top and another the bottom bevil, while the third pair trims the edge of the head. The cutter-blocks, which are worked independently on horizontal slides, are brought into action and kept in position by means of weighted levers keyed to pinions geared into racks. Each cutter-block slide is provided with a small idle roller, which, when it is brought into the cut, presses against a double eccentric keyed on to the vertical shaft which rotates the head. The two eccentrics are at right-angles to one another, so that as one cutter-block is brought into work the other is forced out; thus each set of cutters operates once during one-fourth of the revolution of the head, and, as they revolve in opposite directions, they always work with the grain, insuring a clean cut. This machine turns out heads at ten times the speed at which they can be finished by hand.

The machines used for making the hoops of tight casks are so much like those employed for slack barrels that it is unnecessary to describe them. As in the case of the slack barrel, the insertion of the head is done by hand. Driving on the permanent iron hoops is accomplished by the machine illustrated in Fig. 16. The cask is placed with its hoops on upon a table moved vertically by hydraulic pressure. Above this table is suspended a casting carrying a series of steel driving-arms which are all connected, so that when one is pulled outwards they all open, and are weighted so that when released they fall inwards. When the cask has been placed in position, the table is caused to ascend, and the drivers, catching the hoops, force them into place. To insure each hoop being driven with the proper pressure, a relief-valve is attached to the hydraulic supply-pipe. This valve is closed by a weighted lever, but, as soon as the pressure on the hoop attains the desired amount, the valve opens automatically and the upward motion of the cask is arrested. The advantage of this arrangement is that it insures each hoop being firmly driven on without risk of breakage.

The manufacture of brewers' casks by machinery has not yet become universal in this country. Indeed, the only steam cooperages in England where such casks are manufactured entirely by machinery are at Burton-on-Trent, at Horselydown, and at Poplar; but in Scotland machinery is largely employed for this purpose. There are, however, several steam cooperages for tight and semi-tight casks of various descriptions, among which are those at Hull, at Birmingham, and at Hounslow; and that of the Victualling Department of the Admiralty at Deptford.

In taking advantage of this class of machinery, America is far in advance of this country. At Cincinnati, Ohio, there has long been established an immense cooperage for dealing with tight casks of various kinds, and the Standard Oil Company, having numerous factories in various towns throughout the United States, has for years made its barrels by machinery. On the Continent, also, steam cooperages are rapidly springing into existence. The chief obstacle to the introduction of cask-making machinery is the hostile attitude of the coopers; as many masters, while admitting its great value, hesitate to incur the temporary inconvenience attending the strike which generally follows its introduction. It has now been proved by practical results that, if casks and barrels are to be made economically in large quantities, machinery is necessary; and it may be predicted that before long the hand-made cask will be a thing of the past.

The Paper is accompanied by seventeen drawings from which Plate 5 has been prepared.

[Discussion.

Discussion.

Sir BENJAMIN BAKER, K.C.M.G., Vice-President, said, in these days of keen international competition, the trade of the country could only be maintained by close attention to detail in multitudinous things. Nothing of a mechanical nature was beneath the consideration of the Institution, to which the country at large looked as the leader amongst engineering bodies, to see that Great Britain did not lag behind its competitors in any mechanical detail which might make for the benefit of trade. No doubt members had noticed the statement that ten million casks were required annually for the cement trade. The competition between Germany and England in the supply of cement, even to British Colonies, was well known, and an item such as the cost of the cask would really determine the direction in which the trade would go. Apart from considerations of that kind, members would remember the well-known definition of a "well-informed man"—one who knew a great deal about one subject and a little about most subjects. The thanks of the Institution were therefore due to the Author for enabling a step to be made by its members towards the desirable position of well-informed men, by letting them learn a little about casks. Until he heard the Paper, he must confess to having been rather ignorant on that subject, which had been associated in his mind chiefly with some absurd regulations on the part of Trade Unionists, that if casks were made by machinery, manufacturers should be compelled to have some workmen present to make zigzag marks in chalk round the casks at intervals, for no useful purpose, but that the men so engaged should draw a salary. He had great pleasure in proposing a vote of thanks to the Author for his interesting Paper. Sir Benjamin Baker.

Mr. ALLEN RANSOME thought no person who had not gone through the painful experience of scheming cask-making machinery could have the faintest idea of the immense amount of difficulty which had to be encountered at every step. Looking at a cask, an outsider would say it was the simplest thing in the world; that it consisted simply of a certain number of staves, more or less flat and equal in size; of two heads, which were always supposed to be round; and of six or eight iron hoops. The difficulties commenced at the very first step. The jointing of the staves, Mr. Allen Ransome.

Mr. Allen Ransome. apparently the simplest thing in the world, depended upon the nature of the wood, the class of casks to be made, the amount of bilge the cask was to have, and whether the staves were to be uniform in width or wider at one end than at the other. The two designs of jointing-machines shown in the Figs. represented probably hardly 2 per cent. of those which were either actually in use or had gone out of use. As the Author had mentioned, those which had survived were probably the fittest. Others were to be seen in some more or less perfect works on the Continent, and some were still to be found at work in old-fashioned establishments in England. Another great difficulty was, that it was necessary to pander in every possible way, not only to the prejudices of the trade and the hand-cooper, but also to those of the men who used the cask. If, for instance, a supply of cognac-casks were sent into the Bordeaux districts, there would be no demand for them. Each locality required its own type of cask, having some different peculiarities which the maker of machinery must exactly follow, otherwise the casks would meet with no sale. Take, for instance, the simple claret-cask used in Bordeaux, one of the cheapest casks that could be made for wine. It might be said that anything would do for cheap claret, that any good sound cask would serve this purpose. But that was not so. Claret casks had several peculiarities, which were distinct weaknesses. First, the bilge of the cask was extremely pronounced, the whole of the bend in the barrel being given in about 6 inches length of the stave, which, of course, had the effect, especially with thick staves or when the wood was not very sound, of causing an undue amount of breakage in bending. To remedy that, the staves of Bordeaux casks were usually steamed and pressed before they were jointed. That again necessitated a different class of machines from those exhibited. The claret cask was made with a very long chime, the ends of the staves projecting considerably beyond the heads. The staves were reduced by being shaved off with a long chime, the result of which was to render them weak. If, however, machinery were provided which would produce casks without those weaknesses, there would be no sale for it, on the ground that if claret were supplied in casks which did not look like those to which people had been accustomed, it would be said that the wine contained in them was not genuine. There was a certain amount of truth in this, but it made it difficult for those who had to design machines to meet all these peculiar requirements. Of course, it was known that for years past there had been great troubles in the trade; the strikes, especially

among the London coopers who manufactured barrels and casks for the London breweries, had been notoriously vexatious. Quite recently there had been a strike for an increase of wages. For years past the London coopers had been working on a regular scale, being paid so much for each operation. The Union thought it was necessary to demand a rise of wages, and the masters conceded an advance of 10 per cent. without the slightest demur. No sooner had the men received that, which was supposed at the time to be final, than they formed a new list which was apparently prepared in a very arbitrary way. Instead of being paid about 1*s.* 8*d.* for re-making a hogshead, the price was raised to between 4*s.* and 5*s.*, and other prices were advanced in the same way. One regulation was that every cask was to be stamped with the date at which it was made, and after it had been at work for a certain number of years, whether it was good or bad, it was to be thrown on one side. The masters objected, and a struggle ensued which continued for about four months, and then, as was usually the case, some of the masters began to give way, and virtually the men gained the day. That sort of thing had made coopers and brewers look round to see whether they could not shield themselves from such inconveniences; and some of them had adopted machinery, with the results shown by some of the casks exhibited —two of which, made entirely by machinery, he believed, would take a prize before almost any hand-made cask. The great difficulty in the introduction of machinery in London, was that the brewing trade was so good that the brewers looked upon the cost of the casks as quite a secondary matter; and, although many of them paid extravagant prices for their casks, they did not attach sufficient importance to that, to risk difficulties with their workmen by erecting machinery. That sort of feeling was gradually giving way, and it might be hoped that cask-making machinery would be in future more generally introduced for that purpose. Mr. Allen Ransome.

Mr. Sidney Straker requested the Author to furnish further information as to whether the wire rope tended to slip off the top of the staves, when the windlass was worked, and whether the cask required to be held back. It would be also interesting to know the speed of the machines. It had been stated by the Author that the operation in the ovalling and bevilling machine was performed in one turn of the head. If that was so, he imagined that it would tend to tear the wood in cutting. No doubt that was not the case, but some explanation might be given. Then, with regard to the capacity, he imagined it would be a Mr. Straker.

Mr. Straker. varying quantity, and that, even though gauged accurately before being steamed, the barrel would tend to warp, and so the capacity would vary. Were the barrels always made by gauge, or were they sent out ungauged as to contents? The construction of machinery for cask-making was the result of very many years' application, and no doubt the results were very satisfactory and practical.

Mr. Lewis. Mr. W. B. Lewis observed that whilst the Author had stated that several operations could be performed much more rapidly by machinery than by hand work, he had not given any idea as to the relative cost of machine-made and hand-made casks. A very large brewery firm had been, within recent years, in difficulties, and one could not help thinking that, if there was saving from making casks by machinery, that firm would face trouble with their men, in order to provide a dividend for their shareholders.

Mr. Ward. Mr. Henry Ward found it difficult to understand how the joints could be equally good whether the staves were sawn or planed. The explanation seemed to be that very great pressure was put upon the staves when they were trussed together. Another difficulty was that, in the endeavour to give the proper bend to each one of the staves, it was necessary to steam them, and, apparently, while under the influence of steam or heat the joint was formed. In that case, probably, when the stave became dry again the joint would be somewhat out of truth. He would ask whether that was the fact and whether it gave rise to any trouble. It had been stated by coopers that one reason why hand-made casks would always exist, was that there were no pieces of timber exactly alike, and that a cooper, on noticing the varying qualities of different pieces of timber, could put different bevils upon them, according to their nature. The skill of the coopers in that respect must be great indeed, when it was realised how the whole length of the stave must differ at every point of its length in order to make a tight joint. It rarely occurred that two pieces of timber could be got exactly alike; and of course when dressed by the machines, every piece of timber would be finished to exactly the same shape independent of quality.

Mr. Wynne. Mr. Frank Wynne requested the Author to describe the process of backing and hollowing the staves. Perhaps he would explain the difference between a stave that was backed and hollowed and one to which that operation had not been applied.

Mr. Rigby. Mr. John Rigby wished to call attention to one very important

feature of a cask, namely, the hoop. The cask was nothing without the hoop, which was its backbone. It was evident that, as the cask was made of dry wood, and remembering the purpose to which it was always applied, the hoop of the cask must be submitted to considerable longitudinal stress. He therefore wished to know something about the materials used for cask-hoops, and how they were manufactured; whether in that respect the manufacturer of machine-made casks was entirely dependent upon the manufacturer of hand-made hoops; whether there was any machine which would make the hoops out of sheet-steel by compression and punching; or whether they were welded up from hoop-iron in the good old way formerly practised. It was, of course, important that the hoops should be very firm in order to stand the tension to which they were subjected. Long ago hoop-guns had been made very much in the same way as casks were now made. Some of the earliest of the great ordnances made in the middle ages were designed much on the principle of a cask with a great number of staves held together by hoops. Mr. Rigby.

Mr. E. R. Dolby, referring to the setting-up apparatus, supposed it was improbable that the whole of the staves were made exactly of the same width. He asked, therefore, whether the last one or two staves in a cask were selected so as to fit perfectly tight in the setting-up apparatus. He also asked how the price of barrels made with separate staves would compare with the price of those made all in one piece. There was a process by which barrels were made from one sheet. The trunk of a tree was steamed, and a sheet was taken from the trunk spirally and then nicked out. It would be interesting to know if much had been done in that way. It would seem that very large trunks must be necessary for the purpose, but he believed the plan had been adopted for cement-casks to a considerable extent. Mr. Dolby.

Mr. Henry G. Rawson was connected with a company which was manufacturing in Belgium casks of one piece of wood on the plan which had been alluded to by Mr. Dolby. It was important that the V's or gores in the sheet should not be opposite one another, but as alternate spaces, so as not to split and run together. A log, averaging as a rule 15 or 16 inches in diameter, was taken, and after being boiled three to six hours according to the wood employed—chiefly aspen or poplar—the log was taken out steaming hot, the bark was ripped off it, and it was placed on a cutting-machine and rotated against a knife-blade, with a peculiar oscillating motion which ensured a clean cut. The sheet was then rolled out and cut into lengths varying according to Mr. Rawson.

Mr. Rawson. the size of the diameter of the barrel required, the log having been, in the first instance, cut into a length which would correspond with the height of the barrel. Then, the diameter of the barrel being in that way obtained, the sheet was taken, still hot and damp, to the V cutting-machine where the gores were stamped out, so as to give the barrel the required bilge. The V's might be closer or further apart from one another, by which means a greater or less bilge was obtained. When that was done the sheet was taken to a crozing-machine which cut the ends clean and square, bevilled them, and cut the grooves for the heads. The sheets were then taken to the drying-house, where they were dried by hot air in eight to twelve hours. They were prevented from warping, being fixed down with clamps, and were then ready to be converted into a barrel or to be shipped to another place where there was a factory to turn them into barrels by specially-constructed machinery. That machinery was thus arranged:—First, there was an endless band by which the sheets were taken through the steam-box and moistened sufficiently to enable them to be bent without cracking. Then they were trussed by one or other of the various trussing-machines and hooped, and, finally, a hot-blast was sent through them to fix them in shape, after which the wooden hoops and heads were put on in the ordinary way. That was a rough outline of the method. There had not been much done in England yet, because the necessity for a separate making-up plant at every place where large quantities of sheets were dealt with, involved a greater outlay of capital than the company had at its disposal; but in Belgium and in Germany those barrels had been made to a considerable extent already, and at the present moment the manufacture was being further experimented upon.

Mr. Robinson. Mr. T. N. Robinson mentioned that about twenty years ago his firm had put down a plant for Messrs. Bass and Co., of Burton, for making beer-casks. The process of manufacture did not differ very greatly from that by the machines described by the Author. At the time, the great question was, whether cask-making by machinery would be more extravagant in the use of timber than by hand; and the problem was to produce machines that would joint the staves accurately, and waste as little material as possible. To that end, his firm had introduced a machine for jointing staves which was very similar to that illustrated in Fig. 9, the only difference being the way in which the cutter-block worked. In the latter machine he noticed the cutter-block rose and fell as the stave passed the cutters, whereas in the machine he alluded to,

the cutter-block moved in a lateral direction, being guided by a template and dummy roller, as the stave passed the cutters. The template was bevilled on the edge, and the roller which pressed against it was fixed on a vertical slide which could be raised and lowered by means of a screw; so that as the amount of bilge was required to be altered, it was only necessary to raise the dummy wheel higher or lower on the template, which gave more or less belly to the stave. In that way it was not necessary to change the template to vary the amount of bilge. With regard to jointing-machines in general, he did not think any of those described would make a perfectly accurate joint. The amount of bilge and of bevil would vary in a stave as its width altered, and if the machine was to work accurately, there would have to be some variation in those two points in almost every stave, unless they were all of exactly the same width, which was not usually the case. The general use of machinery for making casks had certainly not been very largely adopted in this country, and he thought the reason was in a great measure that so many processes were necessary. Here there were about twelve different machines for making a cask. That meant a large outlay of machinery for a cooper, and unless he had a big trade to keep them fully employed, it was not likely to pay well. If, instead of having to move the staves and different parts of the casks into so many machines, those operations could be combined into one, and a machine could be produced that would make a cask entirely, there would no doubt be a large demand for such machines. He had seen, some years ago, a machine—the invention of Mr. Dunbar, he believed—in which the operations of chiming, crozing, and setting-up were very successfully combined. In that machine, the staves were set-up on a horizontal collapsible core or drum, arranged something like a double umbrella—as if two umbrellas were mounted end to end on one stick, so that they could be opened and closed together, and outside the frames strips of steel were attached to form a shell. In that machine the staves were set-up round the circumference of the core in its expanded state. Two cones similar to that shown in Fig. 3 were then caused to approach one another by a screw-feed, and in that operation both ends of the cask were trussed at once, the inner core being simultaneously caused to collapse and so to free itself from the staves. The operation of chiming and crozing was performed whilst the cask was held in the cones, which formed a good support for the staves when under the cutters, and enabled the work to be turned out with accuracy and finish. He did not Mr. Robinson.

Mr. Robinson. think the machine had been used for large casks, but for small casks it seemed to be serviceable and quick in operation.

Sir Benjamin Baker. Sir BENJAMEN BAKER thought it would be interesting to have on record some brief description of how the staves were shaped in the old days by hand, and how the hoops were coned. He supposed it was not done by rollers in the early days but by hammer.

Mr. Routh. Mr. W. POLE ROUTH said the hoops were formerly coned by hammer, and the hollowing of the staves was done with the adze. He had had some experience of cask-making on the Continent, where the staves were cut on the block with a large adze which the workman put against his side, lifted up and chopped down, cutting the stave. He would like to say one word about jointing. It had been stated that it was almost impossible for a machine to be made to cut perfectly the chamfer and bilge of the stave. He thought that Messrs. Ransome had brought out a machine some years ago which met every necessity. It shaped the exact form of the bilge and also the chamfer at the same time. If a stave was somewhat wide, it had to have a different bilge and chamfer from that given to a narrower one, and he thought the machine alluded to cut the chamfer and the bilge according to the width of the stave. Theoretically it was perfect, though in practice it was not so. It had two cutters, one on each side of the stave, and the difficulty was to pass the stave through without cutting away too much wood. If the stave was 5½ inches and the machine was set for 5 inches, ½ inch was lost—¼ inch on each side—and the difficulty was to adjust the machine so as to save the wood. It could be adjusted by a hand-wheel, but that would have to be done with every stave, which was impossible in practice; and all the staves had to be sorted into different sizes. He believed the manufacture of the machine had been discontinued on account of the waste of wood resulting from its use. Taking ⅛ inch off each side of a stave, on only a small business, meant a loss of £1,000 or £2,000 at the end of the year. He noticed that Messrs. Ransome had, however, adopted another machine on proper lines, in which they had followed another manufacturer.

Sir Benjamin Baker. Sir BENJAMIN BAKER said it was quite clear that the information on the subject was not yet exhausted, and he hoped there would be some correspondence to supplement the remarks which had been made. With reference to Mr. Rigby's comments on the hoops, so far as he could judge, the riveting seemed very slight, and he should think was not a fifth, or perhaps even a tenth of the strength of the hoop. Mr. Ransome in reply would perhaps have that fact in his mind in answering Mr. Rigby.

Mr. Lewis H. Ransome.

Mr. Lewis H. Ransome in reply, observed that, although several speakers had assumed that all the machines shown on the diagrams were made by his firm, such was not the case. In writing the Paper he had endeavoured to select what he believed to be the best machines at the present date. He had been requested by Mr. Straker to explain how it was that the wire-rope in the process of windlassing did not slip off the cask, and whether it did not tend to pull the cask over. It would be seen on the diagram that the table was close up to the wire-hoop, and therefore the leverage on the cask was not great. The staves, before going to the windlassing machine, were steamed and were quite wet, and the rope seemed to hold better in consequence of that; at any rate, it did not slip at all. With regard to the speed of the chiming and crozing machine, Mr. Straker was correct in supposing that machine finished a cask in one revolution. The cutter-blocks revolved at nearly 4,000 revolutions a-minute, therefore the cutting-speed was very great, and a perfectly clean cut resulted. The only difficulty was that, in doing the chiming in a heavy cask, a great quantity of stuff had to be removed, and consequently the cutters, if not of good steel, were apt to become dull. He had never known a machine that would stand more than twenty brewers' casks without requiring to be sharpened. He was glad that he had been asked about the capacity of the casks made by machinery, because that was the point upon which he claimed that the machine-made cask was particularly superior to the hand-made cask. If a machine was made properly, it must make all casks exactly alike—and consequently their capacity must be the same. The capacity was gauged by the setting-up apparatus. The number of staves should be the same in every case, but the width, of course, varied; the operator had by him a bundle of staves; he fitted in all but the last one or two, and then selected the last one of the right size to fit the two gauge-hoops; consequently they got precisely the same amount of timber in every cask. That being so, and the heads being the same distance apart, and the cask being trussed in a machine which did not vary, the capacity must be invariable. He might mention that everybody who used casks kept a very keen eye on the capacity, and manufacturers would soon hear of it if they did vary. The result of steaming, of course, was to make the cask pliable, and enable the staves to be bent; and he should have mentioned that, after steaming and trussing the staves, many people liked to fire them over a cresset, which, they said, set them, and prevented them from going back. Mr. Lewis had asked about the comparative cost of

Mr. Lewis H. Ransome.

hand- and machine-made casks. That was a difficult question to answer, seeing that there were so many varieties of casks. Taking cement-casks, it might be said that machinery turned them out ten to fifteen times as quickly as could be done by hand. The cost of the machinery was, of course, considerable. About 20 effective HP. would be required to drive a set of machines, and calculations could be made on that basis. For brewers' casks it might safely be said that machine-made casks (even when turned out in small quantities) could be made at at least half the price of hand-made ones. It had been said that the sawn joints were not correct. That was quite true: in neither of the machines described were the joints theoretically correct, but practically they were so; and after the stave had been steamed and subjected to the tremendous pressure put on it in the trussing machine, the joints would never leak. He had seen recently in Messrs. Bass's brewery a machine-made cask with sawn joints which the manager of the cooperage vouched for as being twenty-two years old, and it was not leaking yet. One would not want a better joint than that. A machine referred to as having been made by Messrs. Ransome many years ago, no doubt was theoretically correct. The two cutters-blocks were hung from a centre, which represented the centre of the cask, and opened out radially to the required bevil. That machine did waste some wood, but he had lately seen a machine working on the Continent on the same principle, by which the operator could gauge each stave and set the cutter accurately to the width, insuring no waste, and thus insuring a perfect joint. It was a little slower than the machine described in the Paper, but it produced a theoretically perfect joint. With reference to shaping staves, when undergoing the various operations, he regretted he had not samples of them to exhibit. The stave was first of all, roughly speaking, of equal thickness throughout and rectangular. The operation of jointing was to bevil the two ends. The stave was then rounded and hollowed, which enabled it to be more readily bent. In reply to Mr. Rigby, hand-coopers, and also those who used machinery, bought their hoop-iron ready made. He believed it was rolled, and to be strong enough for its purpose. The thinnest hoops were those used for soap-casks, and they were hardly as thick as ordinary brown paper, but for beer-casks the hoops ran to 14 B.W.G. or even more in thickness. When extra strength was required, two rivets were used in preference to one large rivet, which would weaken the hoop too much. He was much interested by the description of the single-stave cask, known commonly as

Mr. Lewis H. Ransome.

the "Onkin" barrel. He had seen it made in America about seven years ago. It was a fine-looking barrel, but was much more costly than the ordinary cement-barrel. On going over the factory the day it was opened, he was much struck with the enormous quantity of waste wood which he was told had been rejected. If there was any blemish or defect, the piece was cut off and thrown out. He was also told that as the casks were made of one piece, there was not the same tendency to resist warping as in a barrel made up of several staves. A point in which the manufacture of the barrel varied from that described by Mr. Rawson was that the grooves were sawn out by a series of "drunken saws" set on a horizontal spindle, which were out of truth when they entered the staves, and gradually righted themselves as they approached the middle of the stave—so making a V-cut. No doubt, as Mr. Robinson had said, cask-making machinery was not new. On going over the Admiralty Victualling Department at Deptford, he had seen some machines which had been at work over twenty years, with many features almost the same as those now under discussion; but the chief difference was that the new machines would turn out about six times the amount of work that the old type was capable of. The cutter-spindles of those old machines, instead of running at 3,000 or 4,000 revolutions per minute, generally ran at about 500 revolutions. The methods of handling the casks, getting them into and out of the machines and bringing the cutters into action, were all much slower than at present. Far from it being a weak point in cooperage machinery, that every operation was separately performed, the fact was, if an operation was to be done well, it generally had to be done singly. For instance, in making a box there was not a machine for putting the lid on, nailing the sides together and planing it all in one operation, and a cask was a much more complicated thing to make than a box. He maintained that the right plan was to take each operation singly. Of course, it did not pay small makers to have machinery, but for the manufacturer who turned out casks by the thousand, machinery was a necessity. He thanked the members for the attention they had given to his Paper.

[CORRESPONDENCE.

Correspondence.

Mr. Allen. Mr. PERCY ALLEN remarked that while the Paper gave a description of modern cooperage machinery, there were other systems of producing casks and barrels which were worthy of consideration. Barrels of paper and of wood-pulp fibre were now made in America and this country, either by pressing the pulp into exhausted moulds, or by lapping the pulp on metallic cylinders which were afterward collapsed and withdrawn from the pulp-shell. In both processes a seamless cylinder was produced, and in some of the American factories a bulging-form was imparted to the cylinder by placing it in a "bulge-mould" and expanding an india-rubber bag inside it. Square packages could be made by a modification of that method, and were extensively used for the shipment of matches. Barrels or casks made thus required to be indurated or water-proofed, and that was commonly done by treating them with linseed or resin oil, and stoving them at a temperature of about 400° F. Another form of barrel made of paper had lately been produced in Germany, in which the body of the barrel was made up of separate sheets of paper consolidated and rolled under pressure; the bulge being obtained by gores cut out at the ends, with the alternate sheets arranged to break joint. Those casks were now being tried for holding beer; for which use the heads were fixed with metallic lining-hoops and the casks had metallic bung-holes. Paper and pulp-casks, properly made, resisted the action of most chemicals and possessed considerable strength, but their external appearance rendered them unacceptable to dealers in edibles.

Various wooden casks had at one time or other been offered to the public, but as far as he was aware they were only a few that had been much used, viz., the Andrews barrel, the grooved and tongued barrel made in Belgium, the Guelph barrel, and barrels made on the staveless or one-piece system. In the first-mentioned barrel, the staves were grooved-and-tongued, and were nailed together on iron hoops at the place of manufacture, being afterwards transported in the flat form and bent into a circle by a simple form of press worked by hand, the heads being secured by means of lining-hoops. In the grooved-and-tongued casks made in Belgium for holding cement, the grooves and tongues were of V-form. That seemed to produce a satisfactory result, as the casks were used with-

out being papered inside. The Guelph casks were made of sheets cut in America and transported flat, and were rolled up on a making-up machine, two or more layers of sheets being arranged to break joint. In all forms of drum-casks, where it was desired to get the power of rolling and turning them on their centres, a broad hoop was placed round the middle of the drum to give the effect of a bulge. Staveless barrels, or barrels made of one stave of a breadth equal to the circumference of the barrel, seemed to have been first thought of in America about twenty-five years ago. At first the effect of the gores made to produce the bulge was obtained by simply running saw-kerfs down the ends, thus allowing them to be drawn together by the hoops; but now the gores were usually cut out to approximately correct shape either by circular saws or by punches and dies. Most inventors had proposed to make the sheets for those barrels of a continuous piece of wood turned off the log in the form of a shaving—the log being rotated against a broad knife fed continuously towards the centre; and some inventors had proposed to cut the V-grooves and to chime and croze the sheet while it was being unrolled. Mr. Allen.

Having been recently concerned with the re-arrangement of a factory near Antwerp, where the manufacture of staveless barrels and box-board sheets was carried on, he would give a brief description of the process.

The operations were as follows:—The trees were cross-cut in lengths which represented the height of a barrel, or the height of two barrels of smaller size; they were then boiled or steamed. The period of boiling depended on the nature and condition of the wood, and ranged from three hours, for poplar or aspen, to seven or eight hours for beech. The logs were barked previously to being placed in the cutting-machine, so that they were kept hot and the surface was protected from grit. The cutting-machines employed at Antwerp were made on the Onkin principle, and were of peculiar design. Besides the ordinary rotation of the log between the centres and the forward feed of the knife, both the head-stocks carrying the centres and the carriage holding the knife had a parallel oscillating motion. The effect of this was that a double shearing motion was imparted to the knife and pressure-bar, and a very clean cut resulted. The sheet of wood, as it came from the machine, was cut approximately into breadths for the barrel by broad shears, and the sheets were then conveyed to a crozing-machine, which had three cutter-heads running on one spindle. This trimmed the ends of the sheets, cut the crozes, and chimed one or both ends of the sheet. If the three heads were

Mr. Allen. used at the same time, one long sheet might be divided into two smaller barrel-sheets. From this machine the sheets were passed to a V punching-machine, in which the gores were stamped out at each end of the sheet so as to permit it to assume a bulged shape when made up. Those V's were so arranged as to come alternately at each end of the sheet, thus preventing splitting across from one to the other. The punches were made slightly concave, so that they first entered the sheet at the point and the two opposite ends, the shearing action thus proceeding towards the centre. That form of punch effectually prevented any splitting of the sheet, but it was rather difficult to harden and keep in good cutting order, and it would seem better if the punch were made up of separate blades. A varying degree of bulge could be given to the finished barrel by varying the feed-motion of the punching-machine. Other methods of cutting these V's, as for instance with "drunken" saws, had at various times been tried, but the plan by punching was rapid and seemed to answer the purpose. When the sheets came from the V punching-machine they could, if desired, be made up into barrels directly, or they could be dried and stored for future use. At Antwerp the sheets were loaded upon small wooden trucks, with battens between them to allow circulation of the air. The loaded trucks were then passed in order through the dry-houses, which consisted of two parallel sheds, each 70 feet long by 8 feet 6 inches square. A large volume of air was swept through the two houses by an exhausting-fan placed at the end at which the loaded trucks entered, and the incoming air was heated by a system of steam-pipes conveying exhaust steam. The best construction of dry-houses was a matter upon which many conflicting opinions were held; but his experience was, that at all events, for drying barrel-sheets the best results were obtained with a large volume of air passed through once at a comparatively low temperature. The average contraction of poplar, aspen, and alder sheets dried in that way, ranged from 6 per cent. to 8 per cent. across the grain. To convert the dried sheets into barrels, they simply needed to be softened by being steamed, and they then could be bent by hand and headed and hooped in the usual manner. Where large quantities had to be made up, the Oncken Company supplied a small plant, consisting of a continuous steaming-tank and a machine for putting the truss-hoops on, which bent the flat sheets into barrel form, and, by means of two movable bell-machines, forced the truss-hoops on and ejected the barrel. This machine would turn out casks of cement-barrel size at the rate of one a-minute. As they fell from

the machine they were placed on an endless band, which extended from a small exhausting-fan to a coke furnace. The barrels were put on the cool end near the fan, and were taken off near the furnace, and formed a kind of conduit for the hot gases thus drying and setting them. Mr. Allen.

Barrels and casks made by that method were of clean and attractive appearance, and of uniform size, and the flat sheets could be conveniently stored or shipped; but for a factory to produce them economically, it was necessary to develop at the same time a trade in box-boards, lap-boards for cloth and the like, so as to use up all the small pieces of sheeting and defective barrel-sheets. About 10 cubic metres of wood could be converted daily at the works at Antwerp, and the actual cost of manufacture, inclusive of rent, taxes, depreciation, wages and all establishment charges, was nearly double the cost of the wood at the factory. It might be mentioned in connection with the continuous system of cutting timber, that the timber grown in northern latitudes cut cleaner on the side of the trunks facing the north than on the other side. This was very noticeable as the long sheet was being unrolled from the log. He considered that the staveless system was best adapted for barrels not exceeding 30 inches high and 19 or 20 cubic inches in diameter with the bulge and frame $\frac{1}{2}$ inch to $\frac{5}{8}$ inch thick.

19 December, 1893.

ALFRED GILES, President,
in the Chair.

(*Paper No. 2720.*)

"Hydraulic-Power Supply in London."

By EDWARD BAYZAND ELLINGTON, M. Inst. C.E.

THE distribution of hydraulic power has made considerable progress since 1887, when the Author gave to the Institution an account of the works[1] then existing in London. The advance that has been made in those works during the past six years is shown by the following summary:—

——	December 1887.	December 1893.
Miles of mains laid	27	67
Number of pumping-stations .	1	3
Horse-power provided . . .	800	2,600
Power-water pumped in one week	2,062,000 gallons.	7,540,000 gallons.
Number of machines at work .	609	1,925
Capital expended	£150,000	£421,000[2]

In 1887 Hull was the only town besides London in which there was a public distribution of hydraulic power at high pressure. Supplies are now given also in Liverpool, Birmingham, Melbourne and Sydney, while works are being constructed in Manchester, Glasgow and Antwerp. The public distribution of hydraulic power on the accumulator system has thus become well established as a development of industrial enterprise.

[1] Minutes of Proceedings Inst. C.E., vol. xciv. p. 1.

[2] This sum includes an expenditure of £30,000 upon the new pumping-station at City Road in course of construction.

Falcon Wharf and Millbank Pumping-Stations.

The first station at Falcon Wharf, Blackfriars, was described by the Author in the Paper alluded to. In 1887, a second station situated at Millbank, not far from the Houses of Parliament, was in course of construction. The arrangements at Millbank differ somewhat from those at Falcon Wharf. The engines and boilers are of the same type, but the steam-pressure is increased to 100 lbs. per inch. The water delivered from this station, instead of being taken direct from the river, is pumped into tanks from a well sunk to the level of the London clay, 18 feet below Ordnance datum. Overlying the clay is a gravel bed 9 feet 6 inches thick. Headings were driven from the well in the gravel bed for a distance of 300 feet, 150 feet of the heading being under the river. The water pumped from the well is perfectly clear and bright as it runs from the gravel, but rapidly changes its character on exposure to the air, turning to a muddy reddish yellow and precipitating a deposit of iron oxide. The average yield of the well is 300,000 gallons in twenty-four hours, which is sufficient for the present requirements of this station. The water is pumped from the well by hydraulic pumps into tanks over the engine-house. As at Falcon Wharf, filtered-water tanks are placed over the boiler-house at a lower level, the difference of head being utilized to overcome the resistance of the filters to the passage of the water through them. The total capacity of the filtered and unfiltered water-tanks is 300,000 gallons, equal to one day's supply from the station. Hydraulic power is not supplied during the night from this station, but pumping from the well and the filtering-process go on continuously during the twenty-four hours. The hydraulic pumps are not only economical machines but afford great facilities for this class of duty. Their working is quite independent of the running of any particular station so long as the pressure in the general system of power-mains is kept up. Hydraulic pumps (Figs. 1, Plate 6) have now superseded the steam-pumps formerly used at the Falcon Wharf Station.

The methods at first adopted at Falcon Wharf for getting water from the river were not altogether satisfactory. At lowest spring-tides there was a considerable interval during which no water could be obtained from the river, and the storage-tanks on one or two occasions proved to be of insufficient capacity to tide over the interval. It was determined therefore to reclaim a portion of the

foreshore and to sink wells and drive headings under the river in the gravel-bed, as had successfully been done at Millbank. The result was somewhat disappointing. Not more than 10,000 gallons per hour could be obtained, though 280 feet of headings were driven in the gravel on the top of the London clay, and at the termination of the headings there was only about 3 feet of gravel between the top of the heading and the level of the foreshore. The gravel was choked by the river mud. The principal portion of the water used at Falcon Wharf has therefore still to be taken direct from the river. The wells and headings have, however, insured a sufficient quantity of water being obtained at all states of the tide. The reclamation of the foreshore has also given increased area for storage of pipes, which was much wanted, and the use of the hydraulic pumps has effected a considerable economy in the steam-consumption of the station. At the Wapping Station, described hereafter, a well has been sunk to the London clay where the yield is much larger—18,000 gallons per hour—and this without any headings being driven. The gravel-bed at this point is about 12 feet thick.

The arrangements for filtering the water at Millbank are shown in Figs. 2 and 3, the process being that known as the Porter-Clark, but the usual details are somewhat altered to suit the special circumstances. The plant was constructed by Messrs. Gimson & Co., of Leicester, and is capable of treating and filtering 15,000 gallons per hour. The results have been most satisfactory. The preliminary aëration of the water is obtained by allowing the water from the rising-main of the hydraulic pumps to fall over fountains, and most of the iron oxide is precipitated during the time the water remains in the upper tanks. A saturated solution of lime is formed in the lime tank, which contains sufficient for twenty-four hours' supply. The lime is pumped at a regular speed into the softening-vessel, where it meets the unfiltered water coming from the upper tanks. The softening-vessel contains 7,500 gallons of water which passed through it in half-an-hour. This is sufficient time to allow the chemical reactions to take place, and for the excess of lime in the water to be precipitated. The primary object of the process is not to soften the water, so that only sufficient lime is used to carry down the remaining iron oxide, and to form a sufficient filtering medium on the filter-cloths. The water from the softening vessel flows by gravity to the filter-presses (Fig. 3), and the filtered water rises from them into the filtered-water tanks, the difference of level between the clear- and unfiltered-water tanks

being 11 feet. The water, after leaving the presses, passes through a charcoal bed, which strains out any lime that may accidentally pass through them. No lime or deposit has ever, so far as the Author is aware, appeared in the filtered-water tanks. The iron oxide has been very destructive to the cloths. Jute cloths have proved the most durable, but even these require renewing every month. Five filters are in use, and each filter is cleaned every twenty-four hours. The quantity of lime used is 1¼ lb. per 1,000 gallons, which is sufficient to obtain good results from the filters. Incidentally the water is reduced from 24° of hardness to 18° (Clark's scale). The lime-mixer, pump, and the stirrers of the softening-vessel, are driven by a Brotherhood hydraulic engine. A washer is provided for cleaning the cloths, also driven by a small hydraulic engine. Two men are required to look after the plant. These men work during the day only. When they leave work at night all the filters have been cleaned. The hydraulic pumps and the hydraulic engine are left acting, and the whole process proceeds automatically during the night. The night-watchman stops the pumps and engine when all the reservoirs are full. The cost of working the process is, per 1,000 gallons:—

	d.
Lime	0·13
Cloths	0·31
Labour	0·46
Power	0·15
Total	1·05

The output is on the average about 1,500,000 gallons per week.

Wapping Pumping-Station.

It soon became evident that the power distributed from Millbank would be rapidly taken up, and a third station has been erected at Wapping, on a site adjoining the Shadwell Tidal Basin of the London Docks. This station is of much greater capacity than either of those at Falcon Wharf or Millbank, being built to accommodate six sets of engines of 200 I.HP. each. The engines were constructed on the Author's system by the Hydraulic Engineering Company at Chester, and are of similar construction to those used at the other stations, but are made triple expansion with a steam-pressure of 150 lbs. per square inch. The cylinders are 15 inches, 22 inches and 36 inches in diameter severally, each having a 2-foot stroke.

The arrangements in regard to the water-supply are as follows. From the well already referred to, hydraulic pumps deliver the water into two compartments of the tanks covering the boiler-house. The additional water required beyond that yielded by the well is taken by a siphon-pipe from the London Dock. The capacity of the unfiltered-water compartments is 150,000 gallons, and each hydraulic pump can deliver 30,000 gallons per hour. On the level of the stoke-hole floor is a filter-house, in which there are eight filters, hereafter described. A certain amount of settlement takes place in the tanks, from which the water is collected in floating pipes and is conveyed to the filters, and after passing through them is collected in an underground clear-water reservoir in two sections.

The main-engine circulating-pumps draw their supply from the reservoir, and, after passing through the tubes of the surface-condensers, the water is delivered into the remaining two compartments of the tank over the boilers. From these compartments the main engines draw their supply for delivery into the power-mains. It will be understood that it is important to have a head on the suction side of the hydraulic-power pumps. The total capacity of the tanks and reservoirs is 800,000 gallons, equal to about twenty-four hours' delivery from the station. The side-walls and bottom of the reservoirs are of concrete, founded on a bed of stiff clay about 4 feet thick, which overlies the gravel. The reservoirs are covered by brick arches carried on iron stanchions and girders. The spandrils of the arches are filled with concrete, and the whole area is covered by a 6-inch layer of fine concrete.

The top of the reservoir is used for the storage of pipes, &c., and a travelling hydraulic crane is here provided.

The boilers are of the Fairbairn-Beeley type (Figs. 4, Plate 6). Vicars mechanical stokers are fitted to all the boilers. For feeding the stokers, the following arrangement is adopted. The attendant fills a hopper trolly from the coal store, and runs it on the platform of a hydraulic lift, which raises it well above the stoker-hoppers. From the lift it is run along rails above the stoke-hole and is tipped into a hopper, from which creepers distribute the coal to the different boilers. The coal used at the station is discharged from the dock direct into the coal store by a hydraulic crane.

The filters (Fig. 5) were made by the Pulsometer Engineering Company. Their chief feature consists in the method of cleaning adopted. The filtering medium is a bed of animal charcoal. When being cleaned, the filter is open to the drain and the flow of water is

reversed whilst a jet of steam is employed to send a current of air into the filters with the washing-water. The effect of the air is to cause violent ebullition in the midst of the charcoal bed, insuring the thorough cleansing of every particle of charcoal. A screen is provided to prevent the charcoal from being washed away. Very good results have been obtained with this filter. It was not intended to use any chemicals, but the water was not so colourless as that obtained from the Porter-Clark plant at Millbank, though the results were superior to those obtained with the sponge filters as fitted at Falcon Wharf, described by the Author in his former Paper. At Falcon Wharf and Wapping, "alumino-ferric"[1] has been successfully employed to assist the clarification. The quantity used is ½ lb. per 1,000 gallons, and the cost is 0·083*d*. per 1,000 gallons. Each filter at Wapping, with water in ordinary condition, will effectually filter from 4,000 to 5,000 gallons per hour. As at Millbank, pumping from the well and filtration proceed day and night, irrespective of the running of the main engines. As a rule these engines do not run more than twelve hours out of the twenty-four. Variations in the demand for power are met as far as possible at the central-station at Falcon Wharf. This is accomplished by the simple expedient of delivering the water into the mains at the out-stations at a somewhat higher pressure than at the central station. The central station is the only one which runs continuously day and night, Sundays and holidays. It results from this that the output per unit of plant is approximately the same at all stations. At the central-station the work is spread over twenty-four hours: at the out-stations it is accumulated into twelve hours. As regards consumption of coal there does not appear to be much difference from this cause. What is gained at the out-stations during the day through more continuous running is lost during the night with banked fires and steam kept up.

In the Author's former Paper[2] reference was made to a pumping-station at Kensington Court. At the present time this station is only used for accumulators, as the mains have been laid from Piccadilly to Kensington, and the supply there now forms part of the general system. The pressure in the mains laid in Ken-

[1] The following analysis of this substance is supplied by Messrs. Spence & Sons, the manufacturers:—

Al_2O_3 . . .	14·0 per cent.	H_2SO_4 (free) .	traces.
H_2SO_4 (in combination) .	34·0 "	Water . . .	51·4 per cent.
		Fe_2O_3 . . .	0·6 "

[2] Minutes of Proceedings Inst. C.E., vol. xciv. p. 18.

sington Court, formerly charged from that station, is 450 lbs. per square inch; a pressure that was adopted because it was the most suitable for the lifts that were put into the houses on the estate. It is interesting to note, as showing the comparatively great cost of running a small plant when compared with that of working a more extensive system, that it has proved to be more economical to charge the Kensington Court accumulator, loaded to 450 lbs., from the 700 lbs. pressure-mains (thus losing 250 lbs. in pressure), than to work the small station at Kensington Court.

City Road Station.

A fourth pumping-station is in progress at a site on the Regent's Canal, in Wharf Road, City Road, which will have a capacity equal to that at Wapping. The water will be obtained from the canal, and "Torrent" filters will be used, as at Wapping. It is expected that this station will be in running order during the summer of 1894.

Accumulators.

The following accumulators are used in connection with the supply, omitting that at Kensington Court as having no influence on the general system:—

Two at Falcon Wharf . .	20 inches in diameter,	rams	23	feet stroke.
One at Millbank . . .	18	,,	,,	20	,,
One at Philip Lane, E.C.	18	,,	,,	20	,,
Two at Wapping . . .	20	,,	,,	23	,,

The total capacity of these accumulators is about 1,600 gallons. The pumping-capacity of the plant is about 3,500 gallons per minute. These figures show that the accumulators act almost entirely as regulators of pressure, and have a very small influence in respect of storage.

Engine- and Boiler-Tests.

A carefully conducted series of experiments has been made to ascertain the efficiency of the triple-expansion engines and the Fairbairn-Beeley boilers at Wapping. These experiments were made under the supervision of Mr. Bryan Donkin, jun. The duty of both engines and boilers was high, nearly 130,000,000 foot-lbs. being obtained from 112 lbs. of Nixon's coal. The steam-consumption in the engines was 14·1 lbs. per I.HP. per hour, and the boiler and economizer evaporation-test gave 11·1 lbs. of water evaporated per pound of Nixon's coal. The combined

results under the most favourable conditions, *i.e.*, with engine and boiler working up to their full capacity, would give, say, 1·3 lb. of coal per I.HP. per hour (Appendix).

Similar tests were made about the same time with the Millbank engines, when the consumption of steam was found to be 17·71 lbs. per I.HP. per hour, and the evaporation of the Lancashire boilers and economisers to be 9·58 lbs. per pound of Nixon's coal.

The best results for purposes of comparison are stated in the following Table:—

——	Steam Pressure.	Lbs. of Steam per I.HP. Engine.	Calorific Value of Nixon's Coal.	Evaporation Boiler and Economiser, Lbs. Water per Lb. Coal.	From and at 212° Fahrenheit.	Thermal Efficiency per Cent.	
						Engine.	Boiler and Economiser.
1. Millbank. (Compound) .	93·8	17·71	14·45	9·58	11·28[1]	12·24	78·7
2. Wapping. (Triple) . . .	143·0	14·1	15·65	10·22	12·19	15·25	78·2

By the salt test there was no priming either at Wapping or at Millbank. If the amount of water pumped per cwt. of coal at the three stations is compared, the economy of the Wapping plant is very striking. The figures are as follows:—

——	Calorific Value of Coal.	Gallons Pumped per Cwt. Ordinary Small Coal.	Pressure per Square Inch.	Gallons Pumped per Cwt. on basis of equal quality of Coal and equal Accumulator Pressures (732 lbs. per Square Inch).
Falcon Wharf . . .	..	4,558[2]	750	(a). 4,048[4]
Millbank	12·30	4,130[3]	732	4,130
Wapping	11·76	4,920[3]	728	5,118

These are the results of the tests on trial-runs of eight and nine hours with the speed of the engines practically constant. When,

[1] Equivalent to 12·21 with coal of same calorific value as at Wapping.

[2] Durham coal. [3] Midland coal.

[4] The Author had not the calorific value of the coal used here. The figure 4,048 was obtained by using the evaporative result of 10·59 lbs. of water per lb. of coal as given in his former Paper.

however, the results obtained during long periods are compared, the variation is remarkable. The figures are then as follows:—

——	Gallons pumped per Cwt. of Coal reduced to Pressure of 732 lbs. per Square Inch. (b).	Per cent. of Maximum Efficiency $\frac{b \times 100}{a}$.
Falcon Wharf (2 years)	2,425	59·90
Millbank (2 years)	2,663	64·48
Wapping (15 months)	2,885	56·37

This great discrepancy is not to be attributed to any variation in the condition of the engines or boilers. The main cause is in the large amount of heat wasted owing to the intermittent nature of the supply. At each station a small amount of steam is used for purposes other than that of driving the main engines, but this is not sufficient to materially influence the figures. The average efficiency in the case of the Wapping station is less in relation to the maximum efficiency than at the other stations, owing to the fact that the output has, up to the present time, been considerably below the capacity of the plant. As the delivery of power from this station increases the efficiency will increase.[1]

In 1887, the Author gave the result of observations made at Falcon Wharf with engines running at full speed constantly, and under the ordinary variable conditions, which showed that there was an increased coal-consumption of one-third from this cause. That experiment was made under conditions which do not apply to the figures now given; as, though the output of the engines and boilers was constantly varying during the experiment, there was no complete cessation of work. From the records of a consumption of nearly 11,000 tons of coal at the several stations, the Author has been able to obtain the following fairly approximate analysis of the total coal-consumption:—

	Per Cent. of Total Coal.
Coal utilized at efficiency of trials, calculated on total output	60
Coal wasted through intermittent running, based on the experiment at Falcon Wharf referred to	20
Coal used in keeping steam up in boilers and engine-jackets when stopped during nights and Sundays and changing over boilers	12
Steam used for other purposes, variation in quality of coal, defective stoking, &c.	8
	100

[1] The efficiency is at the present time (Nov. 1893) the same as at Falcon Wharf.

Notwithstanding that the total delivery of power has more than trebled since 1887, no diminution of these losses has resulted. Mr. R. E. B. Crompton, in a Paper on "The Cost of the Generation and Distribution of Electrical Energy,"[1] dealt very fully with this question, and showed the overpowering influence of intermittent running upon the economy of production of electrical energy. It is possibly, as pointed out by the late Mr. P. W. Willans, of less importance in hydraulic than in electric transmission, but extended experience in connection with hydraulic-power supply proves it to be the cause of a serious loss of efficiency.

Cost of Production and Load-Factors.

It is commonly believed that the cost varies inversely with the scale of production, but in the Author's view this belief will not stand the test of experience and careful analysis. For each industry there probably is a certain size of undertaking at which the cost per unit of output is practically a minimum; and it is by no means a universal law that with equal efficiency in the conditions of production, the cost is diminished by an increase of the scale of output. The fact is that the economy to be thus obtained is of a decremental nature; and while at first great economy is obtained by an increased scale of output, the advantage soon tends towards a vanishing-point. The Author has given much attention to these questions as affecting the distribution of hydraulic power, and the very interesting and suggestive Paper of Mr. Crompton, already referred to, has led him to think that a publication of the results of the distribution of hydraulic-power supply in London during the past eight years, may be of service in connection with other undertakings involving the supply of energy from central sources.

It has been already shown how large is the amount of waste that arises from the intermittent character of the supply, and it is a matter of the greatest importance, in order to determine the probable limit of the cost of production, to ascertain in what way the load-factor is influenced by increased use of the power. Below are given the London hydraulic power load-factors for the heaviest day's work in each year, 1887–1892 inclusive, and the corresponding load-factors for each year.

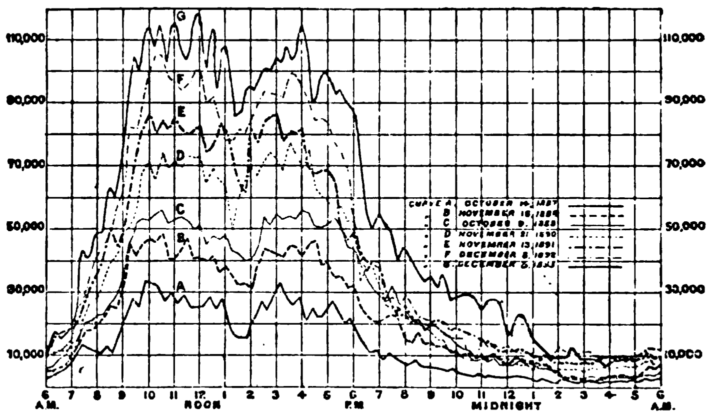
[1] Minutes of Proceedings Inst. C.E., vol. cvi. p. 2.

Year.	Quantity pumped during Year.	Maximum Output.	Annual Load-Factor.	Heaviest Day Load-Factor.
	Gallons.	Gallons per Hour.		
1887	84,647,000	35,000	0·275	0·407
1888	123,055,000	49,000	0·286	0·458
1889	163,883,000	57,000	0·328	0·524
1890	206,421,000	77,000	0·306	0·474
1891	267,671,000	90,000	0·339	0·462
1892	303,032,000	106,000	0·326	0·432
1893	336,636,000	119,000	0·323	0·458

The load-factor is in all cases the ratio of the average output per hour to the maximum output in any one hour during the year.

Fig. 6 gives the separate maximum curves for the different days, which show clearly the great increase in the total power

Fig. 6.



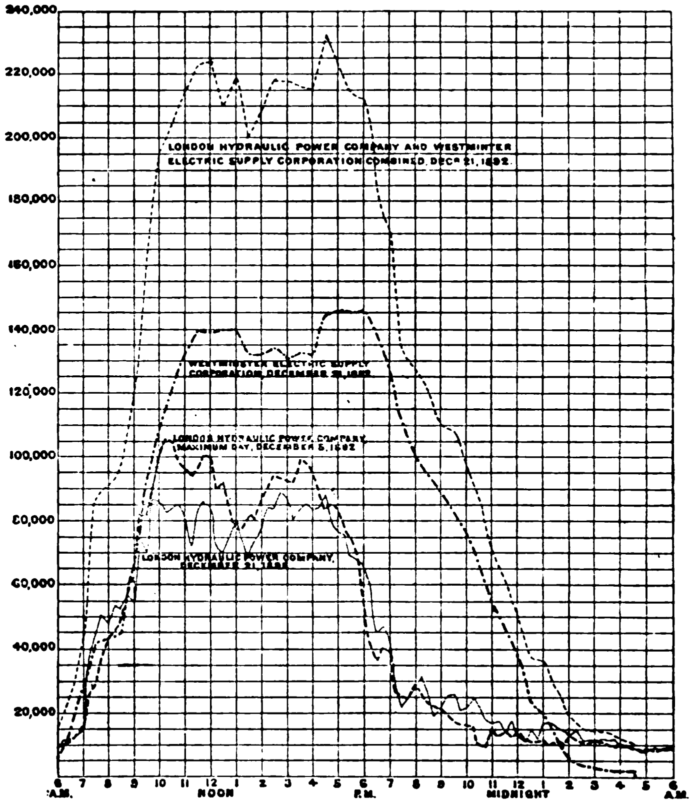
supplied, notwithstanding which the load-factors have not been materially affected. So far, therefore, as regards the influence of load-factors on the cost of production, the experience of hydraulic-power supply does not warrant the supposition that cost is likely to be reduced by the load-factors improving. The increase in the annual load-factors in 1889–93 over 1887–8 is due to the extended area of supply embracing new districts, and not to increase of quantity; the periods of maximum demand for power vary in the different districts and so tend to increase the load-factors. It does not appear possible to make any material improvement in the load-factor by merely increasing the output. It might be supposed, looking at the large amount of plant lying

idle during the night, that, by developing night consumption, say for running dynamos for electric lighting, a much better result would be obtained. Some electric-lighting companies are thus endeavouring to improve their load-factors by supplying power during the day, but the Author's experience is that consumers will not consent to take power exclusively during certain specified hours in the twenty-four, even when a considerable advantage in price is offered; and if they were to do so, no sufficient means exist of keeping them to their bargain. Under these circumstances, at some time or other during the year, it is certain to happen that the time of demand for the power by the class usually served at night, will synchronise with the time of demand of the class served during the ordinary working-hours of the day. The effect of such a conjunction will be to reduce the load-factor of one or other of the two classes of consumption considered separately. For all practical purposes, the only two classes of work open are power during the day and lighting during the night. This is true, irrespective of the particular system employed. It applies with equal force to gas, electricity and compressed-air, as well as to hydraulic power apart from the question of storage. The principal use of the hydraulic system is, at the present time at any rate, for intermittent power, while electric supply is given mainly for lighting.

In order to see the effect of a conjunction of the two, the Author has prepared the diagram, *Fig. 7*, which he has been able to do through the courtesy of Prof. Kennedy, F.R.S. The load-curve of the maximum day's supply of the Westminster Electric Supply Company has been super-imposed upon the hydraulic load-curve of the same day (21 December, 1892), the curves being reduced to the same scale and developed into one. The load-factor of the combined curve is 0·522, whereas the two separate load-factors are,—hydraulic 0·495, and lighting 0·533. The maximum day's output of the hydraulic supply was not on the same day as that of the electric supply, but occurred a few days earlier in the year, on the 5th December. The load-factor for the hydraulic power maximum day was only 0·432. It is noticeable that in the diagram the whole of the supply of the Hydraulic Power Company is assumed to be distributed within the district of the single electric-supply company. The Author has no means of giving the curve of supply of hydraulic power for this district alone, or of ascertaining in any exact manner what the load-factors for the district are, but it is possible to make an estimate with sufficient accuracy. The total quantity

of hydraulic power supplied in the Westminster electric district is about 8 per cent. of the whole. The maximum rate of supply of the total amount of hydraulic power distributed, is only about 73 per cent. of the electric maximum in the Westminster district alone. If the load-factor for hydraulic supply in Westminster were the same as for the whole area, an increase of 5·8 per

Fig. 7.



cent. upon the maximum rate of the present electric supply would furnish all the energy of the hydraulic supply in the district. It is however certain that the load-factor of any single district is less than the load-factor of the whole area supplied. In the Kensington district where hydraulic power is used in private houses and flats, and in some of the large shops, the load-factor for the year was only 0·118 as against 0·326 for

the whole area of supply. Westminster would give a much better load-factor than Kensington, but it is not likely to exceed 0·250, or about the same as the annual electric-lighting load-factor of the St. James' district. A hydraulic load-factor of 0·250 would render necessary an addition of about 7 per cent. to the electric current, to give the equivalent of the hydraulic energy supplied in the district. The load-factor for the Westminster electric supply for the year 1892 was 0·175. If, therefore, all the hydraulic power now supplied were given electrically, the load-factor for the electric supply would only be increased from 0·175 to 0·180. Neither on the maximum day nor on the annual basis is any material effect produced on the load-factors by the combination. The best practicable load-factor is limited in each district by the usual hours of work in regard to power-supply, and by the hours of darkness in regard to lighting. In the absence of storage, owing to the fact that during winter darkness occurs in business hours, conjunction of demand cannot be avoided. In the West End, the maximum period of demand per power is usually during the evening. In many buildings in the West End 30 per cent. to 40 per cent. of the total supply of hydraulic power is used between 6 P.M. and 6 A.M.; and of the total quantity of the London Hydraulic output in 1892, 20 per cent. was pumped between 6 P.M. and 6 A.M.

When dealing with this question of load-factors, it must not be forgotten that they are highly variable in terms and influence, and unless great care is exercised, give misleading results. In cases where a demand for energy, such as for lighting, exists principally during the winter months, there may be for several months a high load-factor whilst for the whole year the load-factor is low. A more uniform output of the same total amount of energy would give a higher load-factor for the year, but would not, therefore, be necessarily supplied on more favourable conditions as regards the cost of production at the station. In the first case, a considerable portion of the plant required to produce the maximum rate of delivery could be thrown entirely out of use during several months of the year, and in this way a high load-factor could be maintained for the plant in use during the whole year. As illustrating this point, it is instructive to compare the fuel-consumption at the Falcon Wharf and Millbank pumping-stations. The load-factor for the former for 1892 was 0·330 and for the latter 0·258. Falcon Wharf works day and night all the year round, while Millbank only runs seventy-two hours a-week. At Millbank, notwithstanding the lower annual

load-factor and the apparently unfavourable conditions, the efficiency in fuel-consumption is 64·48 per cent. against 59·90 per cent. at Falcon Wharf, the efficiency being calculated on the maximum results obtained at each station during the trial. The annual load-factor has, in fact, a far greater influence upon the amount of capital outlay per unit of output than it has upon the cost of fuel and the station-expenses. By breaking up the central-station plant into many sections, it is possible to ensure that machinery actually at work is running usually at as nearly full load as is consistent with the maintenance of an effective and reliable service. Even under the most favourable conditions, a supply approaching the maximum cannot be given for more than 280 days out of 365, and of the 280 (without storage of some kind) it is impossible to get in London from all sources more than twelve hours' work out of twenty-four; so that an annual load-factor of say 0·38 may be regarded as an ideal to which the hydraulic-power load-factor of 0·325 is a fair approximation. Notwithstanding this relatively good load-factor, however, there exists the great discrepancy between the test-run consumption and the actual weekly use of fuel.

This waste can be reduced by sub-division of plant, but it will always be considerable, and the constant running of plant at its maximum capacity, though economical in fuel, may be the cause of increased expenditure on repairs and attendance which will convert apparent economy into extravagance. The supply, moreover, cannot be maintained without reserve plant ready for instant use.

A low load-factor is likely, therefore, to be a permanent cause of low efficiency in all methods of supplying energy which do not admit of a large amount of storage.

It remains to notice the effect of an increasing output in reducing the cost of supply. The experience of the Hydraulic Power Company is shown by *Fig. 8.* The various curves represent the expenditure in pence per 1,000 gallons under the following heads:—

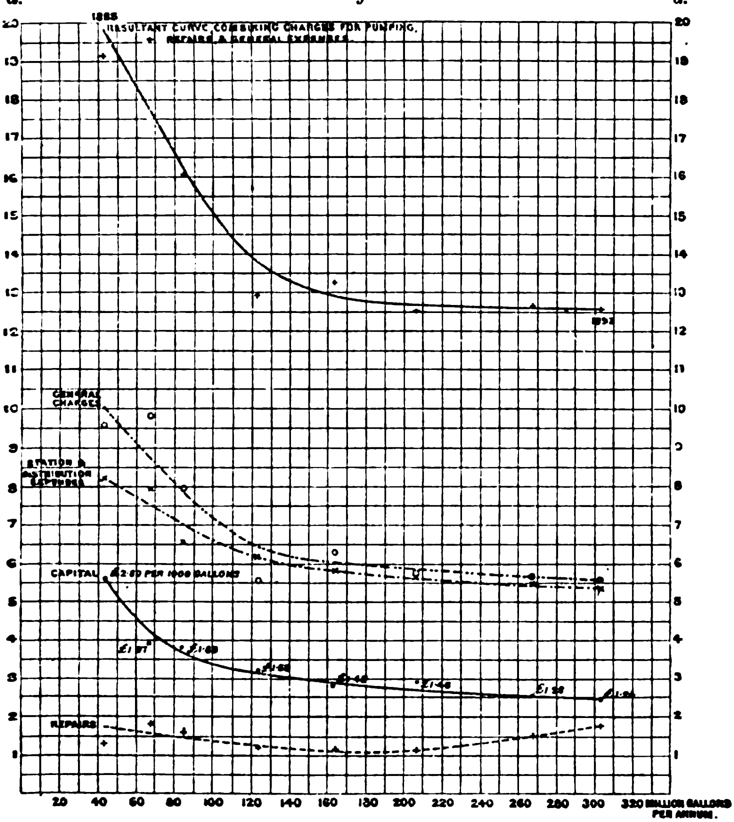
Station and Distribution Expenses—
Wages, coal, water, gas, engine-room expenses and stores. (The coal is assumed to have been purchased at the same price throughout.)

Repairs
to buildings, plant, mains and meters.

General Charges—
Rates and taxes, insurance, office expenses, stationery, incidental expenses, directors, auditors and salaries.

It should be observed that the curves of the station and distribution expenses, and of the general charges, are nearly the same, both in form and in numerical value. The principal reason why the curve for general charges has not fallen below that for the station and distribution expenses, has been the overpowering influence of the increase in amount of rates. The

Fig. 8.



rates paid in 1892 were eleven times the amount paid in 1885, whereas no other single item of expenditure under the head of general charges has increased more than five times, and on the average the increase of the other items has been four times. The total increase in the quantity of water pumped per annum has been nearly eight times during the same period, while the station and distribution expenses have increased about four-and-a-half times.

The curve for repairs is quite normal. At first the plant is by no means fully occupied, and the cost of repairs estimated per unit of output is therefore relatively high. As the quantity of power delivered increases, the cost of repairs per unit is for a time reduced; but as the plant becomes older, the cost increases until the normal cost of maintenance and renewals over a long term of years is reached, when the curve may be expected to become practically a straight line. The amounts annually set aside for depreciation of plant are not dealt with in the diagram, as such sums only serve to average the charges for maintenance from year to year, and the sums so set aside are determined by opinion. Moreover, when the plant to be maintained is very large, and the period under review long, the averaging of the charge for maintenance from year to year becomes nearly automatic, and a depreciation fund cannot well be distinguished from an ordinary reserve.

The Author would have been glad to give similar curves for the Hull hydraulic-power undertaking, which has now been in existence for sixteen years; but as this was impracticable, he thought it would be of interest to give the curves shown in *Fig. 9*, in which the ordinates indicate the percentage of total cost to receipts. Though this method is not strictly comparable to the previous analysis of the London results, the curves point to the same conclusion (as the scale of charges is practically the same in London and Hull), notwithstanding that the Hull undertaking has only one-twentieth of the capacity of that in London.

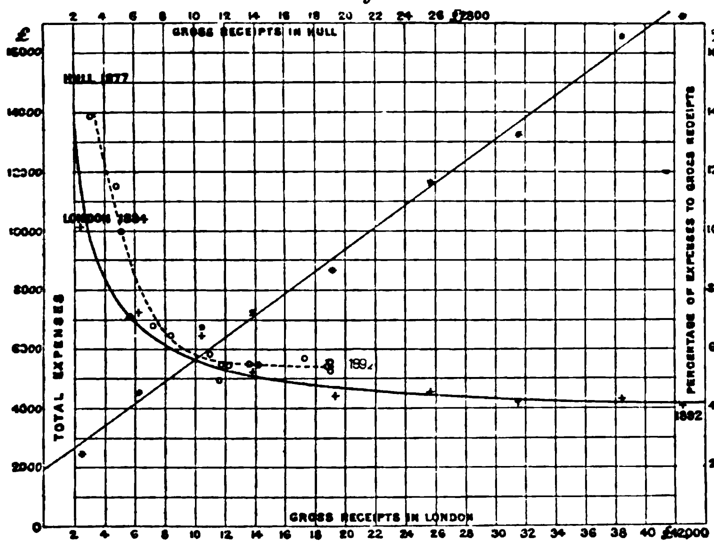
The influence of capital expenditure on plant in proportion to output, is obviously of as much importance as the actual expenses, in determining the cost at which the consumer can obtain his energy. Here the conditions are much more variable. In *Fig. 8* is given the curve of the capital per 1,000 gallons delivered expended in London on the hydraulic-power supply. An important factor governing the curve is the outlay on pumping-stations relatively to that on mains. Hitherto, the ratio has remained fairly constant. Eventually, the outlay on mains should become less, and the curve of capital would continue to droop. It is evident from the *Figs.*, as a whole, that there is small probability of power (including charges for depreciation and interest) being profitably supplied much under current minimum rates.

The diagonal straight line across the diagram, *Fig. 9*, shows in a striking manner the way in which the expenses grow in direct proportion to the increase of business. The stars on each side of the straight line indicate the gross expenditure in relation to the gross receipts taken from the published balance sheets of the

London undertaking. The straight line is a mean line between these points. It will be noticed that the diagonal line cuts the left boundary of the diagram at 2,000, and indicates approximately the minimum cost of working the undertaking irrespective of the amount of output. This minimum cost remains a constant charge on the undertaking, and its amount determines the percentage of reduction in the cost of working as the output is increased, and also the length of time or amount of increased output required to reduce the cost of working approximately to a minimum.

There is no reason to believe that experience in connection with hydraulic supply in London is likely to be materially

Fig. 9.



different from that of other undertakings established for supplying energy in towns from central artificial sources. The general form of the curves will probably be the same. The numerical values only will vary in particular cases. A very important element of the numerical value is, in all cases, the scale on which the works are originally planned. If works are planned on a moderate scale in relation to the probable output, and will allow of extensions as required, the minimum cost of supply will be approximated to within a comparatively short period; and under existing institutions and methods of business, further development is unlikely to exercise any material influence upon it.

It is assumed that due care is taken throughout to avoid waste of all kinds, and that at each stage of the enterprise there is practically no opportunity for what is called cutting down expenses without sacrificing efficiency.

Applications of the Power.

Since 1887, there have been few new important applications of hydraulic power. Progress has been made in the private use of the Greathead high- and low-pressure ejector fire-hydrant. Some public buildings have been fitted with them, notably, the National Gallery and New Scotland Yard, and a few private buildings such as Queen Anne's Mansions. It is still to be regretted that no public use is made of the sixty miles of main in the most important districts of the metropolis, where valuable property is collected, by providing a thoroughly adequate and reliable fire-extinction service such as cannot possibly be obtained from the ordinary hydrants attached to the Water Companies' mains alone. A few Pelton water-wheels, specially designed for high-pressure supplies, have been satisfactorily at work for some time past, and constitute a great improvement upon the hydraulic engines hitherto used. The Pelton Company assert that the high-pressure wheel will give 80 per cent. efficiency. This has not so far been realized here in practice, though an efficiency exceeding 70 per cent. has been obtained. By mounting the motor on the dynamo shaft, there can be no doubt that 66 per cent. of the hydraulic energy can be utilized as electrical energy. Taking the rate of power supplied at 2*s.* per 1,000 gallons, the cost of the electrical energy obtained in this way works out at about 6*d.* per Board of Trade unit. It is possible, therefore, that there may be considerable development of the use of the hydraulic power for this purpose. At the present time, the electric lighting of Antwerp is being established upon a combined hydraulic and electric system devised by the late Mr. van Rysselberghe of Ghent. Hydraulic power is obtained at the central station by steam plant as in London, and is conveyed through pipes to various sub-stations in the city, being there converted by means of turbines and dynamos into electrical energy for distribution through the network of conductors. The Author expresses no opinion on the merits, or otherwise, of the arrangement, but he understands the efficiency of Mr. van Rysselberghe's turbine is no greater than that of the Pelton wheel. The Pelton wheel, at any rate, is an extremely simple, economical and convenient apparatus.

Taking all circumstances into account, there does not seem to be any reason why the use of hydraulic power in London should not in the future continue to increase as it has done in the past; and there are good grounds for holding the opinion that the hydraulic method of transmitting power will be able to maintain the position it occupies at the present moment, as the most effective and economical mechanical system in existence for the supply of energy for intermittent power-purposes in large cities.

The Paper is accompanied by ten drawings, from which Plate 6 and the *Figs.* in the text have been engraved.

[APPENDIX.

APPENDIX.

TRIALS OF A TRIPLE-EXPANSION COMPOUND VERTICAL SURFACE-CONDENSING STEAM PUMPING-ENGINE AND FAIRBAIRN-BEELEY BOILER AT THE WAPPING PUMPING-STATION OF THE LONDON HYDRAULIC POWER COMPANY.

—	1 March 7th, 1892.	2 March 11th, 1892.	3 March 18th, 1892.	4 March 25th, 1892.
Duration of trial . . . . . . hours	9	8	9	9
Engines.				
Diameter of high-pressure cylinder . in inches	15	..	..	..
" " intermediate cylinder "	22	..	..	..
" " low-pressure " "	36	..	..	..
Stroke . . . . feet	2	..	..	..
Total revolutions	31,972	28,235	32,815	49,006
Average revolutions per minute . .	59·76	58·82	60·77	90·70[1]
Barometer . . . inches of mercury	29·8	29·4	29·9	Not taken.
Vacuum . . . " "	28·0	27·74	28·43	
Total I.HP.	179·80	185·63	206·55	
Thermal efficiency of engines, per cent.	14·25	14·73	15·25	..
Boilers.				
Total heating-surface . square feet	800	..	..	..
" grate area . . . " "	20	..	..	..
Pressure of steam in lbs. per square inch	118·3	142·8	143·0	141·3
Temperature of issuing gases ° Fahr.	..	..	557·5	..
Temperature of issuing gases before passing economiser. . . ° Fahr.	355·7	345·3	328·1	436·2
Temperature of issuing gases after passing economiser. . . ° Fahr.	247·3	253·8	238·5	271·3
Amount of condensed steam from jackets . . . . lbs.	2,479	2,620	2,674	..
Coal.				
Quality	(Inland)		(Nixon's Navigation.)	
Total weight fed on to grates . lbs.	3,287	3,200	2,774	3,744
" " of clinker, etc. . "	114	196	237	162
Fuel burnt per hour (including clinker, etc.) . . . . lbs.	365·2	400·0	308·2	416·0
Fuel burnt per square foot of grate "	18·26	20·0	15·41	20·8
" consumed per I.HP. per hour "	2·03	2·155	1·49	..
Calorific value of dry coal . . . . lbs. of water per lb. of coal	13·75	11·76	15·65	15·5

[1] Two engines running, one at 55·4, and the other at 35·3 revolutions per minute.

TRIALS OF A TRIPLE-EXPANSION COMPOUND VERTICAL SURFACE-CONDENSING STEAM PUMPING-ENGINE AND FAIRBAIRN-BEELEY BOILER AT THE WAPPING PUMPING-STATION OF THE LONDON HYDRAULIC POWER COMPANY.—*contd.*

—	1 March 7th, 1892.	2 March 11th, 1892.	3 March 18th, 1892.	4 March 25th, 1892.
Feed-Water.				
Feed temperature on entering economiser. . . . . . . ° Fahr.	67·4	66·3	72·2	63·5
Feed temperature on entering boiler . ° Fahr.	204·7	203·2	194·1	215·8
Heating-surface of economiser . . . square feet	1,375	..	..	..
Total water evaporated . . . lbs.	26,199	23,005	28,365	41,584
Water evaporated per lb. of fuel (wet) lbs.	7·97	7·19	10·22	11·11
" " " " " (dry) from and at 212 (calculated) lbs.. .	9·96	9·27	12·25	13·44
Thermal efficiency of boiler, including economiser . . . . . per cent.	72·5	78·9	78·2	86·7
Amount of feed-water per I.HP. per hour (engine) . . . . . . lbs.	15·14	14·59	14·10	..
Steam-pipe condensation and loss[1] per I.HP. per hour . . . . . lbs.	1·05	0·9	1·16	..
Total feed-water	16·19	15·49	15·26	..
Total quantity of water pumped, gallons	160,800	140,600	160,880	235,228[2]
Accumulator pressure . . . . . . lbs. per square inch	727·3	728·5	795·0	800·0[3]
Water pumped per cwt. of coal consumed . . . . gallons	5,530	4,920	6,495	7,036

[1] Long range of steam-pipes for several engines. The condensation alone was about 0·5 lb. per I.HP.

[2] Calculated at 4·8 gallons per revolution.

[3] Approximately.

[DISCUSSION.

Discussion.

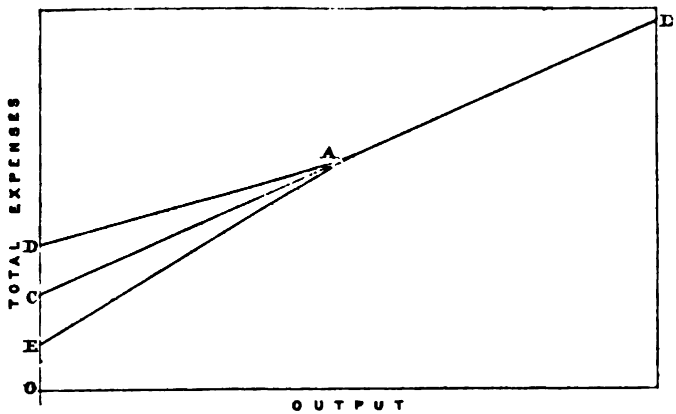
Mr. Giles. Mr. ALFRED GILES, President, said that the cordial thanks of the Institution were due, and he was sure would be heartily accorded to Mr. Ellington for his valuable Paper.

Sir Frederick Bramwell. Sir FREDERICK BRAMWELL, Past-President, asked whether the hardness of the water at the Millbank station—24° before the softening process and 18° after—was hardness calculated according to Dr. Frankland's scale, or according to Dr. Clark's scale?

Mr. Ellington. Mr. E. B. ELLINGTON replied that it was calculated by Dr. Clark's scale. In 1887 the size of the largest mains was 6 inches internal diameter. The mains used for conveying the supply from the Wapping and City Road Stations were 7 inches internal diameter. He wished to explain further the diagonal line in *Fig. 9*, showing the growth of expenses. He had mentioned in the Paper that the minimum and permanent charge upon the undertaking was indicated by the point where the diagonal line cut the left-hand boundary of the diagram, all increase in the cost being in constant proportion to the increase of the output. That statement was open to some misconception if taken in a general sense. He did not mean to say that, as a general rule, the point where such a mean line of expenses would cut the boundary would indicate in fact the minimum charge. He believed it was to a large extent accidental that the straight line in the diagram happened to so nearly correspond with the facts up to the boundary. To make the point clear he had prepared *Fig. 10*. It would be no doubt admitted that whatever might happen in the earlier stages of an undertaking of the kind in question, a time would arrive when, other things being equal, the increase of expenses would bear a constant ratio to the increase of output. Let the line A B in the diagram (*Fig. 10*) represent the cost in relation to output during such a period. A B produced would cut the left boundary at C. It was very unlikely, however, that O C would represent the actual minimum cost of working. The actual minimum cost would be greater than O C, say O D, or less than O C, say O E, and, instead of following the normal line C A B, the actual line of expenses in relation to output would be either D A B or E A B. The area C D A would represent an increased cost above the normal, and the area E C A a diminished cost below the normal, during the earlier years of the respective under-

takings. The scale on which works were started was the principal element affecting the question. It followed that a great relative reduction in cost per unit as the output was increased was a sign of a great loss. The greatest economy was obtained when the actual line of expenses in relation to output was at starting below the normal line, and that involved the necessity for the cost of each amount in output being greater during the earlier years than the normal. He had said that the line C A B, plotted as mentioned, would cut the boundary above the origin O. That would be true in nearly all cases; if it were not true, it would indicate that there were causes in operation increasing the cost per unit, irrespective of the amount of output. But if the different items of expense, as shown in *Fig. 8*, were plotted in Mr. Ellington.

Fig. 10.



a similar manner, it would be found that some of those items would cut the horizontal line. When that was the case, it indicated an increasing charge per unit. Of course, in the case of such items, the actual cost would necessarily be above the normal during the first few years, and the curve of cost per unit would be irregular. The curve on *Fig. 8* for repairs was an example.

Professor KENNEDY said there were three points in the paper of particular interest. The first was illustrated by *Fig. 8*, in which the Author had been able to show how, for his own particular work, a maximum size of station was comparatively soon reached, beyond which no increase of size would diminish the cost of production. In that case this maximum was apparently reached in a station containing about 1,200 HP., and it was clear that the same thing must happen in other similar enterprises, such as Prof. Kennedy

Prof. Kennedy. those connected with electric lighting. Professor Kennedy did not think the maximum size was at all so soon reached in electric-lighting stations, it was more probably reached at 4,000 or 5,000 HP.; but there was no doubt that at some power, which he believed to be between 3,000 and 5,000 HP., it was about as economical to use more than one station as to concentrate all the power into one station. The load-factor had been referred to by the Author, who had evidently come to the conclusion, with which all would agree, that the cost of production was comparatively little affected by load-factor if that most useful expression was used in the sense intended by the Author. Comparing *Figs. 6* and *8*, for example, it would be seen that from 1887 to 1892 the load-factor had remained practically constant, whereas the expenses were reduced to about two-thirds of their original amount. There was, however, a kind of load-factor which did affect the working costs very much. The cost of production depended not so much on the total amount of work done as on the extent to which all the units at work were fully loaded. For instance, noting the actual work done by any engines, pumps or dynamos in the twenty-four hours, and at the same time the total quantity of work which would be done by those same machines if they had been running at full load for the same number of hours as that which they had actually run, they would then get a ratio which might be anything from 50 to 85 or 90 per cent. That ratio really did very greatly affect the economy of working. It meant the proportion of full load to which the machines were on the average loaded, and that proportion very much affected the economy. With 1,200 HP. per station, they might be running most economically with only 200 HP. if it happened that it was just full load for one of the units. They would be running less economically with 250 HP., because that was the full load for one and a quarter the load for another unit; whereas the 250 would give a higher load-factor than the 200 in the sense in which that expression had been used in the Paper. In *Fig. 7* the Author had added together the output of an electric-lighting station and of an hydraulic-power station, and had shown practically that one could not be used to help the other to make up the load,—that both wanted their loads at the same time. The diagram further showed that hydraulic-power plant had altogether an advantage over electric-lighting plant. The day chosen by the Author was in December when electric-lighting work was heavy, but if he had taken any day at another time of the year, instead of the curve obtained, he would have had a very inferior one, while the

Prof. Kennedy.

hydraulic curve remained always of the very broad shape year in and year out. He had been referring to a former Paper, in which the Author had given a diagram of the work done every week for a number of years, and it appeared there that the seasons had practically very little effect on the output; whereas in electric lighting the seasons had so much effect that a foggy day in December would give twice as much work as a bright one in November, and four times as much as an average day in August. It was interesting to note that the shape of these load curves would have value to the historian of the future as annotations on the social habits of the people. They would see, for instance, how the demand for water always dropped off just before dinner-time. At seven o'clock the hydraulic curve went down to its minimum, whereas electric light was used till it was time to go to bed. There was a curious thing about the lighting curve which might have been noticed, viz., that at about half-past seven there was a change in the curvature, and the line got a little less steep. That meant that the shops were shut about that time. After that the curve went on with private houses and hotels only. The first very rapid decline was due to the closing of the shops which took place about seven o'clock. In his analysis of the coal consumption, the Author had given a statement which Professor Kennedy considered exceedingly valuable. He had never seen the matter treated quite similarly before, although he had worked out somewhat similar points in connection with water-consumption. The figures given by the Author seemed to show that in a hydraulic-power station worked with particular regard to economy, the actual total consumption was 66 per cent. greater than the coal consumption got by the same machines working at full power on special trials. It was further shown where the difference was lost. The only thing he would suggest was that probably the second item, which the Author had marked loss by intermittent running, included also the loss due to the running at less than full power; although he knew that in hydraulic stations intermittent running simply meant running more slowly. In electric-lighting stations the corresponding quantity would certainly cover the loss which occurred in this way, because the engines and machines were constantly running at much less than full power.

Mr. Porter.

Mr. J. H. Porter had not had the pleasure of conferring with the Author upon the matter of the works with which his name was associated, in the preparation of the Paper, but he might say that what he had read was very clearly, succinctly, and happily given, and did ample justice to the action of the apparatus. He

Mr. Porter. was very pleased to find that during the years it had been in operation it had answered the Author's expectations so satisfactorily. With regard to Sir Frederick Bramwell's question as to the hardness of the water, of course it was exceptionally hard compared with the ordinary water-supply of London; but as might be seen in the report on the domestic water-supply of Great Britain, in which Dr. Frankland had taken a very active part, it was found that in most of the shallow wells in and about London the hardness was exceptionally great owing to infiltration. He had an analysis of that particular water. The exceptional hardness was due to sulphates of lime and soda. There were about 13 grains of carbonate of lime, $2\frac{1}{2}$ grains of carbonate of magnesia, $4\frac{1}{2}$ grains of sulphate of lime, $4\frac{1}{2}$ grains of sulphate of magnesia, $4\frac{1}{3}$ grains of sulphate of soda, nearly 26 grains of chloride of sodium, and between 3 and 4 grains of oxide of iron per gallon of water—the latter substance being for several industrial purposes more objectionable than all the others. The total amount of those "solid matters" found in solution in the water, which at its first issue from the soil was perfectly bright and clear, was more than 58 grains per gallon. The Author had been careful to mention that his primary object was not to soften the water. Had he required it for evaporation in steam-boilers, it would have been so—in the sense that softening was the result of ridding it of the mineral matters it contained—which would otherwise have been deposited as scale in the boilers, pipes and valves. To completely soften it, it would have been necessary to employ a large quantity of lime and some soda, as was done at Penarth and other stations on the Taff Vale railway for the feed-water of locomotives.[1] He remarked that, in the apparatus at Penarth and in that of which the plant at Millbank furnished an example of a somewhat different kind, the object being the application of what was known as "Clark's process" in such a way as to admit of large quantities of water being dealt with in a small space. That explained the position of the filter-presses upon the raised gallery. It was usual to place them on the ground-floor, and it was only the exigency of restricted space that had led to the softening-vessel being carried in the horizontal position above instead of being as usual set vertically on a base of brickwork. By bringing the water-softening plant to that higher level, the water issuing from the filters had, of course, less distance to ascend to the tanks above. The advantage of the system of working under pressure was more apparent

[1] Minutes of Proceedings Inst. C.E., vol. xcvii. p. 354.

where, as is often the case, the plant was upon the ground-floor, and the store-tanks or high-service cisterns were situated 40 or 50 feet higher. There were many applications of that kind, some of which had been adopted under the advice of members of the Institution for locomotive and other supply. In another kind of apparatus filtration was dispensed with by increasing the capacity of the softening-vessel so much that the water, having longer time for its passage through it, parted with the carbonate of lime by precipitation and subsidence. Mr. Porter.

Mr. HENRY DAVEY observed that the Author had pointed out that it was not an accumulator system, although the word accumulator appeared frequently. It was interesting to observe that the total power accumulated did not exceed 15 HP. for one hour. The accumulators, therefore, were only regulators, but, being placed in suitable situations, they doubtless tended to equalise the flow of water in the mains and thereby to somewhat reduce the frictional loss of head in the pipes. One hundred gallons of water per minute delivered into the mains at the accumulator-pressure of 732 lbs. per square inch was equivalent to 50 HP. The number of gallons pumped per cwt. of coal on a trial, and also the number of gallons pumped on the average of fifteen months' working at Wapping, had been stated by the Author. Reducing the figures to pounds of coal per effective HP. per hour, the result showed 2·6 lbs. on the trial, and 4·6 lbs. as the average of fifteen months' continuous running. The value of any reduction which might be made in the 4·6 lbs. by improvement in the load-factor would depend on the relative cost of coal to the other expenses. It would be seen that the station-expenses (of which coal formed probably the largest item) were about one-half the total expense—amounting to 5½*d.* per 1,000 gallons, or 0·67*d.* per effective HP. per hour, the total expenses amounting to 1½*d.* per HP. per hour. It would also be seen that the "stand-by" consumption of coal was only 12 per cent. of the total consumption, and probably one-quarter or one-third of that was due to maintaining the steam-pressure in the engine-jackets and steam-pipes; so that the portion of the "stand-by" loss due to banking fires was inconsiderable. It was stated in the Paper that the Pelton wheel was superior to the rotative hydraulic engines previously in use. There appeared to be no satisfactory water-pressure engine in use which would vary the consumption of water with variations of load. Water-pressure rotative engines, as a rule, were made with valves and passages far too small. Quick-running engines wasted power in rapid changes of the direction of flow of the water, and Mr. Davey.

Mr. Davey. generally by throttling. A long-stroke slow-running hydraulic engine would give 70 to 80 per cent. efficiency when working with a constant load. With regard to the engine trials, he observed that the Author used the term "thermal efficiency" of the engine in a way which was not unusual, but which was nevertheless ambiguous. He thought that engineers should agree on a more accurate way of expressing thermal efficiency. The word "efficiency" meant, to his mind, the nearness with which the actual came to the theoretical or possible. He knew of no theory that the total heat of steam was available for conversion into mechanical work; on the contrary, it was taught that a large portion of the heat was not available. Therefore the expression "thermal efficiency" as used in the Paper was misleading in two ways—firstly, in making the steam-engine appear a worse heat-engine than it really was, and, secondly, in making it appear that theory had taught what it had not. Joules' experiments gave no warrant for using what was known as his equivalent in the way he had mentioned, and he thought the time had come for a rational expression to be agreed upon by engineers. Turning to Willans' engine trials, engines would be found giving a thermal efficiency of 50 to 60 per cent., whilst the Author's engine only gave 15 per cent. Why? Because "thermal efficiency" did not mean the same thing in the two cases.

Prof. Unwin. Professor W. C. UNWIN said the comparison of load-lines made by the Author was both instructive and accurate. He would, however, point out that the electric-lighting curve taken gave nearly a maximum demand on the electric-lighting station at noon. That was not an ordinary electric-lighting curve. Of course it was true that if a power supply and an electric-lighting supply were combined, a very large plant might be required to meet a possible maximum demand; but if it was meant that the ordinary or average load-line would not be improved because the power supply and the lighting supply were combined, the conclusion drawn would be probably not the one intended, and certainly it would not be accurate. After many years' experience, the Author had come to the conclusion that there was small probability of power being profitably supplied under the minimum rate of the General Hydraulic Power Company of London. It was stated that there were a few Pelton water-wheels on the London Hydraulic Supply, and in connection with that it would greatly add to the interest of the Paper if the Author would say how many HP.-hours they consumed per annum. Professor Unwin imagined that it was an insignificant part of the whole power

supplied. With great fairness and accuracy, the Author had computed that, taking the rate of power supplied at 2*s.* per 1,000 gallons, the cost of the electrical energy obtained by using motors in London on the Hydraulic Power Company's mains worked out to about 6*d.* per Board of Trade unit. That calculation was fairly made, and might indeed have been put slightly lower. 6*d.* per Board of Trade unit for power supplied to electric lighting was equivalent to £56 per effective HP. per year of 3,000 working-hours. The power supply from the hydraulic main, as given by the Author, if applied for ordinary power purposes for 3,000 hours in the year, would cost £56 per HP. per annum, which was, he said, the lowest price at which the power could be supplied on that system. The Author's final conclusion was that the "hydraulic method of transmitting power will be able to maintain the position it occupies at the present moment, as the most effective and economical mechanical system in existence for the supply of energy for intermittent-power purposes in large cities." On that Professor Unwin had two remarks to make. The word "intermittent" was, he feared, rather undefined. In the sense in which the Author used it, he entirely agreed with his conclusions, but a factory engine which worked ten hours in the day only was intermittent-acting; but it was not power used in the way which the Author was contemplating. By intermittent power the Author meant distinctly power which was used for a few minutes and then stopped for a longer period, and was then used for a few minutes again. For intermittent power in that sense, used for a few minutes at a time, and then stopped for a longer period, it was quite true the hydraulic-power system was successful, and he did not know of any other system which when lifting-machinery alone required to be provided for, was more economical, or by which it was likely to be superseded. But if people were limited to having power supplied in towns at £56 per HP. per annum there would be no very large amount from central stations used for ordinary purposes. The cost would be prohibitive of its use, and he was unable to assent to the view that there was no other way in which power could be distributed in towns effectively and at a cheaper rate. Prof. Unwin.

Mr. Bryan Donkin had enjoyed the privilege of making several trials at Wapping and other stations, and could confirm the even working of the pumping-engines. He had also made several jacket-trials at Wapping, with one, two and three jackets in use, and without jackets, and combinations of two. The general result was that the total gain from all jackets to no jackets was about Mr. Donkin.

Mr. Donkin. 10 per cent. That was with a triple engine consuming about 14 lbs. of steam per I.HP. per hour. It must be remembered that in that particular case there was only about 33 per cent. of the total internal surface jacketed, which was the key to the question of jacketing. The ratio of the internal cylinder-surface jacketed to the total internal surface in any particular engine was important. An engine was often said to be jacketed, but that statement was of no value unless the extent of the hot surfaces was known. In the trial mentioned, the low-pressure jacket gave a much greater gain than both the other jackets together. Those jacket-tests would be published shortly. He thought they ought to be very grateful to the Author for his interesting Paper.

Mr. Barry. Mr. J. Wolfe Barry, Vice-President, asked the Author what his experience was as to the most economical pressure for working hydraulic plant. The pressures in the mains described, he gathered, was 700 lbs. per square inch, as the ordinary working-pressure. Had that been found to be the best, or had any data been obtained which would indicate a higher pressure as being likely to be hereafter employed? In his experience, higher pressures were used often for working hydraulic plant, and he did not quite know whether the 700 lbs. had been arrived at by any exact series of experiments, or whether it was the power that was thought to be the best at the time the plant was put down as the result of experience then available. Of course it was obvious that an increase of pressure must result in some increase of expenditure in the pumping-engines. Whether that increase of expenditure could be economically undertaken or not was a matter which, perhaps, the Author could give some information upon. He noticed that the people at Kensington got their supply at 450 lbs. pressure per square inch instead of 700 lbs., and he should like to ask whether they paid the same money for the less efficient water; because of course in hydraulic engines the point to be considered was the quantity required to fill the cylinders or rams, and certainly there must be a much larger quantity used with 450 lbs. pressure per square inch than would be used with 700 lbs. Apart from the last matter mentioned, he should very much like to have some guidance as to what was the best pressure to aim at in new hydraulic plant.

Sir Frederick Bramwell. Sir Frederick Bramwell, Past-President, desired to ask one other question and to make a remark. Had the Author tried whether the pressure of 700 lbs. per square inch prevailing in the mains had any effect on the rate of flow through the pipes as compared with the case where there was a much lower pressure, the "heads" being in the two cases equal? He was not suggesting that it had;

Sir Frederick Bramwell.

but frequently an engineer in charge of an undertaking like this had opportunities of making experiments which other engineers had not, and he should like to know whether the experiment had been made, if so he hoped the result would be communicated. With regard to Professor Kennedy's remarks, he could not help thinking that Professor Kennedy had in his mind engines which were non-condensing, the kind which were commonly used in electric-supply stations, he was sorry to say. The non-condensing engine when not working up to its full power, whether it was a three- or a two-cylinder engine, was no doubt then a comparatively uneconomical machine, but if it were a condensing-engine, that same want of economy, in his experience, did not appear when it was lightly loaded. And when it was remembered that engines delivering pressure-water no doubt always condensed their steam, he should have thought the efficiency of the engine would have varied very little indeed even with considerable alteration in the load. It was known that the Cornish pumping-engines—not that he spoke of them as being good engines—were regulated by the cataract to make from three or four strokes per minute up to fifteen or sixteen, and he thought it would be found that they did as good duty at four strokes per minute as at fifteen or sixteen, or nearly so. Therefore, giving all due weight to Professor Kennedy's argument, he thought he should be right in saying that the Professor's remarks were based upon his large experience with non-condensing engines.

Mr. Deacon.

Mr. George F. Deacon asked if the Author would be good enough to give some information with respect to the sponge filters described six years ago in the Paper previously referred to.[1] He would also like to understand exactly what was meant by the statement that it was important to have, in that class of engine, a head on the suction side of the pump. He did not see that it was so in the hydraulic pump described, but to whichever class of pump the Author referred, it appeared to him that if it related to the question of priming of the pumps in intermittent work, that need not be a difficulty. For instance, in the hydraulic pumps he had designed for the intermittent work of the Mersey Aqueduct Tunnel and of the Vyrnwy Aqueduct Subway under the Manchester Ship Canal, the engines worked only for a short time, perhaps twenty minutes or half an hour in the course of eight or ten days. They started automatically, and before starting the pumps were automatically primed; the motive power for priming and turning

[1] Minutes of Proceedings Inst. C.E., vol. xciv. p. 1.

Mr. Deacon. on the water from the Vyrnwy main being the rise of water in the sumps at the bottom of the shafts in which the pumps were placed. With this arrangement no difficulty whatever had been experienced. The other question upon which he should like to have a little explanation was that of the method which had been adopted for drawing water from the Thames ballast at Millwall and at Falcon Wharf. It appeared that something like 300,000 gallons had been obtained daily by certain headings at Millwall, and only 10,000 gallons a day at Falcon Wharf. He thought that was a very common experience; no one could predict what might happen under such circumstances, and in his experience such methods came to grief ultimately in one of two ways—either the gravel became silted up, or the water came in too quickly at particular places, and was not filtered. He would like to know not only the lengths which had been given to the headings, but the diameters, the method of lining and the way in which the weep-holes were arranged. A very large example of that kind of failure had occurred at Florence. Most of the water of that city was pumped from brick-lined headings of large size situated in the gravel below the bed of the Arno. Pumping proceeded until the water in the headings was shallow, and the infiltration, which was at first abundant, appeared to have steadily decreased until it was inadequate to the requirements, involving further outlay in extensions. The original cost of that undertaking was very great, but the quality of the water was not entirely satisfactory; and it appeared to him that, owing to the constant silting-up of the gravel with the mud of the Arno, periodical extensions and consequent further cost would be necessary.

Mr. Segundo. Mr. E. C. DE SEGUNDO was struck by the similarity of the load-curve of hydraulic-power supply to the load-curves of electric-supply stations. That circumstance, unfortunately, rendered the concurrent supply of hydraulic and electrical power from the same generating station of no advantage as far as an improvement in the load-factor was concerned, as, instead of filling up the depression in the electric load-diagram, the addition of hydraulic-power supply increased the maximum demand on the engines. The diagram, *Fig. 8*, suggested a comparison between electrical and hydraulic-power supply. The head corresponding to 732 pounds per square inch was 1,683 feet. One horse-power therefore represented a consumption of $\frac{33,000}{1,683 \times 10} = 1 \cdot 96$ gallons per minute. The cost of 1,000 gallons delivered was $12 \cdot 6$ pence, according to *Fig. 8*; and for the purposes of comparison it was necessary to

know the average rate at which these 1,000 gallons were pumped. Mr. Segundo. The consumption for 1892 was stated to be 303,032,000 gallons. From that it would appear that $\frac{303,032,000}{365 \times 24 \times 60}$, or 576·7 gallons were pumped on the average per minute. That corresponded with 294 HP., whereas 2,600 HP. were provided (December 1892). That was a very low load-factor. But even assuming that the same load-factor would exist though the company should be called upon to supply 1,000 gallons per minute, it was reasonable to conclude from the curve of total expenses in *Fig. 8* that the cost would be slightly diminished, say to 12·5 pence. Therefore the cost of 1,000 gallons delivered in a minute might be put down as 12·5*d.*

Now as to the comparison with electric-power supply:—One Board of Trade unit was equivalent to 157·6 gallons per hour. 1,000 gallons at 732 lbs. per square inch consumed in one hour represented $\frac{1,000}{157 \cdot 6}$, or 6·34 kilowatt-hours. The total cost of the hydraulic equivalent of 6·34 kilowatt-hours was therefore $\frac{12 \cdot 5}{6 \cdot 34} = 1 \cdot 97$ pence. There was certainly no electric-supply company using steam-power able to approach that figure. The question that naturally suggested itself was—wherein lay difference between hydraulic- and electrical-power when both were generated by steam-power? There was no great difference in the process of generation; if anything, the sum of the efficiencies of the apparatus were against hydraulic compared with electrical power. The efficiency of engine and boiler should be the same in both cases. The efficiency of the pumps should be less, if anything, than the efficiency of the dynamos. The efficiency of distribution should, within a reasonable area, not be very different. The load-factor was practically the same. Taking a pound of coal and following the various transformations of its energy until the lamp terminals of the consumer of electric-light were reached, an interesting result was arrived at. A pound of good coal might be assumed to have a calorific value of 14,500 thermal units, 10,150 thermal units might be assumed to be transmitted through the boiler-plates to the steam, 1,015 thermal units might be allowed for stand-by losses, leaving 9,135 thermal units; 15 per cent. of that, or 1,370 thermal units, might be assumed to be utilized by the engine; with a reasonable load-factor and with suitable generating units, 75 per cent., or 1,028 thermal units, might be assumed to be converted into electric energy at the dynamo terminals, and a further 15 per cent. might be allowed for

Mr. Segundo. losses in distribution, &c. There was then left over 874 thermal units. Now 3,600 thermal units were equivalent to one kilowatt-hour, so that 4·12 pounds of coal should correspond to one kilowatt delivered. The efficiencies assumed were low; but in spite of that there was no electric-supply company whose coal-consumption had reached that figure per kilowatt-hour delivered in daily work. In reference to Mr. Davey's criticism of the Author's definition of "thermal efficiency as applied to steam engines," Mr. de Segundo thought that as a basis of comparison that definition was preferable to that suggested by Mr. Davey, because it took for a standpoint a definite figure, whereas Mr. Davey's suggestion involved a comparison with a relative basis, which, though perhaps fairer for an individual engine, did not seem to correspond so well with the practical definition of efficiency—namely, the relation borne by receipts to expenditure.

Mr. Giles. Mr. GILES, President, before calling upon the Author to reply, remarked on one thing that struck him very forcibly, viz., the increase of the assessment to eleven times the original rate. He believed the Author's Company was a Limited Joint Stock Company, and when the business was undertaken in 1885 it did not pay such good dividends as it was doing now. Was not that, perhaps, one of the causes for the increase of assessment?

Mr. Ellington. Mr. ELLINGTON, in reply, said that no doubt the incidence of rates in London upon such undertakings did to a large extent depend upon the profits earned. At the same time it was perfectly clear that whatever was paid in rates was part of the cost of the undertaking. They were probably let off easily at first and were now rather severely dealt with. In reply to Mr. Deacon's question as to the headings. A shaft was first sunk to the level of the London Clay, and the heading was then driven through the gravel overlying the clay. A concrete bed was laid in the clay, and the sides of the heading were built in perforated brickwork, the top being a solid brick arch. The headings were 3 feet 6 inches high by 2 feet 3 inches wide. During the operation of driving, the gravel was kept up by perforated timber, which was left in, and the space between the boards and the brickwork was carefully filled with coarse gravel. There was no doubt that, in the case of the headings under the river at Falcon Wharf, the small yield was due to the great amount of mud and clay in the gravel which prevented the in-flow. At Wapping, where the gravel was much thicker, and the distance from the river was greater, the flow was very considerable, and it was found that 18,000 gallons per hour were obtained without driving any headings at all. A

question had also been asked as to the filters that were used at Falcon Wharf in which sponge was the filtering material. Those were still in use, but their efficiency was certainly not so high as that of the charcoal-filters which had been described in the Paper. The sponge-filters at Falcon Wharf were originally supplemented by charcoal-filters, and since then other filters, similar to those shown in *Fig. 5*, had been added to increase the filtering capacity of the plant there. As to the necessity for a head on the suction side of the pumps, he had referred to the main engine pumps pumping water into the accumulators and mains. Those pumps ran occasionally at a speed of 250 feet, or 63 revolutions per minute, and the head on the suction had a material influence in enabling this high speed to be maintained without shock. With regard to the analysis of the coal-consumption, in which 20 per cent. was shown to be loss by intermittent running, he never had been quite able to satisfy himself as to the precise cause of its being so great; but it seemed to be much the same at all the stations, and to occur under all conditions of manipulation of the stoking. It clearly was due to running the engines very much below the maximum speed at which they were run during the trials. Referring to the figures in the Appendix, it would be seen that the fuel-consumption, even with comparatively small differences of speed, did vary considerably. The load on the hydraulic engine, as Professor Kennedy had pointed out, was nearly constant; it was only the speed that varied. He did not agree with Mr. Davey with regard to the standard for thermal efficiency. It seemed to him that the standard used in the Paper in giving the results of the trials of the engines was the right one. It was the whole heat taken from the boiler by the engine in the shape of steam. This heat was intended to be used in the engines as far as possible, but only about 15 per cent. of that heat was converted into useful work. He was inclined to think—in fact, Mr. Davey himself admitted it—that the only reason for proposing to alter the standard was because he seemed rather ashamed of its being such a small amount; but that surely was not the question at all. Figures were wanted for comparison, and if it was believed that that heavy loss could not be avoided, it was no use hiding the fact and pretending the heat was not lost. The comparison was not only between steam-engines and steam-engines, but there were other heat-engines which came into competition with steam-engines —the gas-engine, for example. It would not be desirable to have a special standard for every special engine in order to show appa- Mr. Ellington.

Mr. Ellington. rently higher efficiency. He took a similar view with regard to the questions of the load-factor and losses of coal. He believed that the load-factor had great influence. Like the loss of the heat in the condensation in the steam-engine, it was one which, under the circumstances, could not be got rid of. With regard to the "load" diagram, *Fig.* 7, for the purpose in view it was essential to take a maximum day for electrical supply; because the point to determine was what amount of storage or reserve plant would have to be kept in order to supply both power during the day and light during the night. It did not at all affect the question whether a different kind of diagram resulted at different times in the year; it was the time in the year when the maximum occurred that had to be considered. He felt that there was the question, how far the annual load-factor might be affected; because it was clear the annual load-factor of the ideal supply, about one-third of the whole, was nearly double the annual load-factor of electric lighting, which in Westminster was only about one-sixth of the whole. Therefore, if it were possible to supply double the amount, one-half for power and the other half for lighting, then the load-factor would be affected to the extent of half the difference between the two separate load-factors. The demand in London for power was so small in comparison with that for light, that a combination was not likely to produce any material effect one way or the other. Professor Unwin had referred to the cost of converting hydraulic energy into electrical energy. Taking the rate of 2*s.* per 1,000 gallons the hydraulic energy worked out to 3*d.* per HP. hour, or £37 10*s.* for 3,000 hours' work. The minimum rate of supply in London from the power mains was 1*s.* 6*d.* per thousand, or £26 per 3,000 HP. hours. There was nothing in the figures he had given in the Paper to suggest the probability of a lower cost than this. At such rates hydraulic power might be used electrically. He had been making some experiments with the Pelton wheel dynamo. When his experiments were completed he should have pleasure in giving the results to the Institution. As to Mr. Segundo's interesting figures bearing upon the cost of producing hydraulic and electrical energy, he wished to repeat what he had said on other occasions—that on *à priori* grounds there did not seem to be any reason why there should be much difference in cost between them. At present hydraulic power was much the cheaper; but he was confident that in process of time the two systems would be more nearly in line in regard to cost; so long, however, as steam was required, he expected hydraulic power would retain its lead.

Mr. Ellington.

With regard to pressure, that nominally maintained in the mains throughout London was 700 lbs. to the square inch. The actual pressure supplied was about 750 lbs. The only reason for adopting 700 lbs. pressure was because it was that selected by Lord Armstrong when he introduced the system of hydraulic power. The consequence was that there were already in existence many plants working at about the pressure of 700 lbs. on a square inch; and for commercial reasons it was very desirable to adopt a pressure which would be able to supply the needs of those already using hydraulic power without any alteration of machinery. It had the further advantage that the methods of dealing with such a pressure were well known. There was a considerable number of makers of machinery who were able to supply the public at once without further knowledge than that they already possessed. Experience had fully confirmed Lord Armstrong's views with regard to the suitability of such a pressure for general supplies. For lifting-purposes, it was extremely useful. The difficulties increased if the pressure was considerably higher, but there were special classes of hydraulic plant under which it certainly was most desirable that higher pressures should be employed; for instance, for the machinery, with which Mr. R. H. Tweddell's name was associated. He had been accustomed to use pressures of 1,500 lbs. per square inch, and that pressure had been of great service in that class of machinery. He further concurred in what he believed was Mr. Tweddell's view, that the pressure might be yet increased with advantage. Then there were processes where, of course, much higher pressures were desirable. There was no impediment in the use of a main pressure of 700 lbs. per square inch for any purpose, but it involved an introduction of additional machinery in the shape of intensifiers. In Manchester the pressure adopted was 1,120 lbs. on a square inch for the public supply. The reason for its adoption was a commercial reason. One of the principal applications expected in Manchester was for pressing cotton, and the principal cotton-pressing establishments in the city, especially the more recent ones, had been so arranged as to start with an initial pressure of about half a ton on the square inch. The accumulators were loaded to that pressure, and the lifts worked, and the presses were used with the same pressure. It was only the final squeeze which was given direct by the pumps or by intensifiers. At Glasgow, the corporation believed that there might be a considerable use for the hydraulic power in connection with pressing-work of that class. The intention was that the pressure should be 1,000 lbs. per

Mr. Ellington. square inch. As a general rule the higher the pressure the greater economy in working. In Glasgow there was a very large use of power rom the Corporation waterworks mains already. Some 600 machines were at work, and there was very great disturbance of pressure in the mains in consequence. The idea was to get rid of the very uncomfortable demand that existed upon the mains in the City by introducing the high-pressure supply for power purposes. The charge for power supplied on the Kensington Court Estate was on a special scale. In reply to Sir Frederick Bramwell's question whether experiments had been made with regard to the flow of water in pipes at high pressures, he was sorry to say he had made no such experiments. It was a matter often in his mind, and he would take seriously into consideration the question of conducting such experiments. He had hitherto used the ordinary formulas for flow which applied to lower pressures.

Correspondence.

Mr. Anderson. Mr. E. W. Anderson, referring to the Author's remarks as to the generation of electricity by means of the Pelton wheel, desired to know the efficiency of that machine, or rather, of a combination of Pelton wheel and dynamo, at loads ranging from a light load up to full load; also what regulating arrangements, if any, there were for keeping the efficiency as constant as possible, in other words, regulating the use of the water. He presumed the calculation of 6*d.* per unit was based upon full power. Now, in ordinary electric lighting, it was obvious that the full load would rarely be attained, and it seemed, therefore, that the calculations of cost should have been based upon the efficiency at the average load, and not at the full load. He would also like to know how far the fluctuations of pressure which occurred in hydraulic mains would affect such a plant working incandescent lamps. He imagined that a variation of pressure which was of little moment in working lifts, cranes, etc., might be of considerable importance in working dynamos for electric lighting, unless some automatic pressure-regulator was adopted, either for the water or for the dynamo. One other point was, the cost of 6*d.* per unit appeared to have been calculated at the terminals of the dynamo. There was, however, loss in the mains, and interest and depreciation on the capital expenditure, both in the mains and in the wheel and dynamo, which he thought ought to be allowed for. It might not amount to much, but still there must be something, which the consumer

had to pay for. He inferred from the low rate quoted (2*s.* per 1,000 gallons) that the Author had calculated upon a considerable consumption of water; or was it a special rate proposed for that particular work, irrespective of the quantity used by each consumer? Mr. Anderson.

He would be glad to know whether the Author had experienced any trouble with the joints of the cast-iron mains. He understood that in the hydraulic installation at the Royal Arsenal some trouble had been experienced from corrosion on the inside of the joints, owing, apparently, to the use of the somewhat brackish Thames water. It appeared as if the machined ends of the pipes, where not protected by the gutta-percha joints, had frequently corroded very much.

Mr. C. Humphrey Wingfield would like to know what arrangements the Author had found best for drawing water from the river direct, without choking the suction-pipes and pumps with sand or floating weeds, &c. That was a difficulty which always presented itself when it was necessary to provide circulating water for the condensers of steam-engines near a river. He observed that the diameter of the intermediate cylinder given on page 223 was 22 inches, whereas it should have been 23¼ inches, if taken as a mean proportional between the two other cylinders. The point of cut-off was not given, but if it was nearly the same in each cylinder he should expect the effect of that proportion would be to prevent the HP. cylinder from doing its fair share of work. He would be glad to know if that was so in the case referred to, as although recommended in a standard work on marine engineering, it was not, in his opinion, based on correct reasoning. Mr. Wingfield.

Referring to *Fig. 6*, the uppermost point in each curve agreed with column 3 of the Table on p. 230, with the exception of curve "E," and he would be glad of an explanation of that point. Roughly, all the curves in *Fig. 6* were similar, but drawn to different scales. Probably the average curve for one day would present the same general form. In the latter case, the ratio of the maximum height to the mean height of the average curve would give the load-factor, and the fact of the curves in *Fig. 6* being so nearly similar, showed why the load-factor was so slightly affected by the total output. *Fig. 7* showed, as might have been foreseen, that in a district where the demands for both electric and hydraulic power were limited to almost the same hours of the day, viz., from 10 or 11 A.M. to about 6 P.M., a combination of the two would not perceptibly improve the load-factor. If, however,

Mr. Wingfield. the hydraulic power could be limited to a district in which offices and warehouses required it between those hours and the same engines could be employed to deliver electric energy to the West End, where those who had been using hydraulic energy in town during the day would employ electric light in their houses at night, the case might be different. Those who used the hydraulic energy at present provided during the day left their places of business at night, and to reduce the combined load-factor shown in *Fig. 7* the energy should follow them in another form. He supposed it was out of the question to attempt to store half a day's supply of water at the pressure or the equivalent head required in the service-pipes; but any step in that direction would tend to equalize the power required from the engine, and would materially reduce the working cost, if the interest on the outlay did not overbalance the gain.

The plan of breaking up the central-station plant into sections was a very necessary one for any works in which the power varied greatly. He was glad to see that the Pelton wheel had given such satisfactory results. Had any experiments been made to determine the probable total energy per pound of water issuing from an orifice of known size at the end of a pipe of known length and diameter? For instance, if it were desired to erect a 15-HP. wheel 50 feet from the main, what data were available for determining the necessary diameter of supply-pipe and nozzle?

Mr. Wingfield had never seen a trustworthy method of measuring the pressure in a moving column of water, although some experiments with fire-hoses had been lately published in "Engineering" which bore on that point. Many years ago it was proposed to measure the speed of ships by means of two vertical glass tubes passing through the bottom of the vessel. Their lower ends were bent at right angles, the one pointing forward, the other aft. The inventor expected the water in the former tube to rise and the latter to fall, when a suitable scale would give the speed of the vessel. It was afterwards found that in an unbent tube passing through the bottom of a vessel, the water fell even lower than in the one bent backwards, none of the three pipes giving the same head as that outside, and that was what happened whenever a current of water or air flowed nearly across the opening to a pressure gauge—the gauge indicated too low a pressure. The pressure inside the nozzle being unknown, how was the theoretical power computed, from which the 80 per cent. efficiency given on p. 238 was reduced? The pressure, when the jet was closed, was of course useless for the purpose.

He suggested the following method of testing the energy in a jet without risking the error due to incorrect readings of pressure. Mr. Wingfield.

If the temperature was 53·3° F., 1 lb. of water was equivalent to a prism 3·3077 feet long and one square inch in section.

Hence $$V = 2 \cdot 3077 \frac{W}{A},$$

where V denotes feet per second; W, lbs. per second; A, area of section of stream in square inches.

Then $$\text{foot-lbs. per second} = \frac{V^2}{64 \cdot 4} \times W = 0 \cdot 082697 \frac{W^3}{A^2};$$

or $$\text{foot-lbs. per minute} = 4 \cdot 96172 \frac{W^3}{A^2}.$$

Therefore the HP. of jet was nearly $\dfrac{W^3}{665\ A^2}$.

It was only necessary therefore to catch and weigh the water which issued in 5 or 10 minutes from a suitable nozzle and estimate the rate of discharge. That and the area of the jet were all the data required. If the nozzle was properly formed, there should be no perceptible contraction after the jet left it. Where there was a *vena contracta*, the nozzle might be considered as not completely filled by the jet, and the formula would give too low a result unless the section of the stream was measured at its narrowest part.

If desired, the constants could readily be corrected for temperature, supposing that to differ materially from 52·3° F. He was rather surprised to see no mention of the Rigg hydraulic engine, having been under the impression that it was specially adapted for hydraulic-power supply.

Mr. Ellington, in reply to the Correspondence, stated that experiments wlth the Pelton wheel and dynamo were in progress; therefore at present he was unable to give results. So far as they had gone, they indicated that an efficiency of 66 per cent. could be obtained at full load. The efficiency of the Pelton wheel varied only slightly at different loads and constant speed, and, therefore, the combined efficiency at varying loads depended principally on the construction of the dynamo. Interest and depreciation on the special capital expenditure would have to be added to the hydraulic power cost, but if the dynamo were used to supply single blocks of buildings there would be no additional cost for mains. The rate mentioned of 2*s.* per 1,000 gallons was the usual charge of the Hydraulic Power Company for con- Mr. Ellington.

Mr. Ellington. sumptions of 500,000 gallons per quarter; and for very large consumptions, 1*s.* 6*d.* per 1,000 gallons was the usual charge. No special rate had as yet been fixed for hydraulic power used for driving dynamos. At 1*s.* 6*d.* per 1,000 gallons, 6*d.* per Board of Trade unit would be about the total cost of electricity obtained in this way, including maintenance and interest. As regarded maintenance of the hydraulic mains, no trouble had been experienced. Many of the mains had been in use for ten years, and from various causes single pipes in those mains had from time to time been renewed, but no corrosion had been observed. The principal cause of leaks had been settlement in the roadways; and a leak once set up quickly enlarged the aperture with very curious results, at first sight suggestive of corrosion, but which on examination were found to be due to mechanical and not to chemical action.

In answer to Mr. Wingfield, the arrangements for drawing water from the river direct were described in the previous Paper.[1] The I.HP. of three cylinders in the engine-trial marked No. 1 (Appendix) were respectively, high-pressure, 58·18; intermediate, 64·47; and low-pressure, 57·15.

During the trial marked No. 3, the horse-powers were 70·47, 72·07, and 64·01 respectively. The cut-off was adjustable only in the high-pressure cylinder.

The maximum output during one hour given for 1891, viz., 90,000 gallons, was quite correct. The maximum hour's work in that year did not occur on the same day as the maximum supply during any twenty-four hours between 6 A.M. and 6 A.M.; though, as a rule, the maximum hour's work occurred during the maximum day's work.

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[1] Minutes of Proceedings Inst. C.E., vol. xciv. p. 1.

SECT. II.—OTHER SELECTED PAPERS.

(*Paper No. 2698.*)

"Experiments on the Condensation of Steam in Cylinders of Iron and other Metals."

By BRYAN DONKIN, M. Inst. C.E.

THE Author has considered that some further experimental investigations[1] on the important subject of the condensation of steam were desirable; especially to ascertain, if possible, how far the temperature of the surfaces touched by the steam affect the results. He has also wished to make a comparison between the steam-temperature ranges and the amounts of steam condensed. In an engine, before the piston moves, the steam has to fill the passage and clearance spaces, and this it does at constant volume in a certain time. During this time, no doubt, the greatest amount of condensation takes place, and it is probably almost instantaneous, as the steam is at its maximum, the walls are at their minimum temperature, and the conditions are favourable for rapid exchanges of heat.

In the experiments described in this Paper, indicator diagrams were taken, and the number of lbs. of water condensed per hour per square foot of internal surface was calculated. The cylinders used were of cast-iron, phosphor-bronze and glass. The temperatures of the metal cylinder-walls were taken, also the range of temperature of the steam, and the quantities of steam used were weighed. The ratio of the cylinder-volumes to the internal surface touched by the steam was varied, and also the number of fillings per minute. Experiments were made, working both condensing and non-condensing. No piston was used, as it was desired to obtain the effects of condensation apart from power. With this exception, all the other conditions were similar to those of slow-running steam-engines. Generally the cylinders were filled 35 times per minute, and steam of 50 lbs. pressure per square inch used; the cut-off occurring at about $\frac{1}{5}$th to $\frac{1}{7}$th of the time of the steam-stroke. By this arrangement there was no expansion of the steam. The weight of water produced from condensation

[1] Minutes of Proceedings Inst. C.E., vols. xcviii. p. 250; c. p. 347; cvi. p. 264.

under the different surface-conditions was determined, and also the weight of steam shown by the diagrams. After the cut-off, the percentages of the water and of the steam in the mixture, could be determined. Thus the effect of time alone, in diminishing the pressures of the steam by condensation, for a given set of surface-conditions, could be ascertained. The indicator diagrams resembled in shape those taken with a moving piston, where the vertical lines are proportional to the pressures in lbs. per square inch, and the horizontal lines to the feet per minute passed through by the piston. Working without a piston, the verticals give pressures as before; but the horizontal lines represent time only, in parts of a second, when the diagrams are developed for equal times. These areas may, therefore, be called "second-lbs." in contradistinction to the usual "foot-lbs." in power diagrams. The drum of the indicator was driven in the ordinary way from a radius-rod. The tests being made with small cylinders, the results will not bear comparison with those of ordinary steam-engines, but they are comparable with each other. When the surface-temperatures are mentioned, the internal surface touched by the steam is referred to.

It may be well to consider the full meaning of the condensation of steam of 50 lbs. pressure per square inch, corresponding with a temperature of 297° F. in an ordinary engine, and the importance of keeping every part of the metal touched by it, at this or a higher temperature. Apart from the question of power, condensation is produced by contact of the steam during a certain time with surfaces at a lower temperature than itself. It should be the object of the engineer to reduce this condensation to a minimum, or to obtain from a given weight of steam the maximum area of diagram. The less the area of surface exposed for a given volume of steam, and the less the time of exposure, the less the condensation. Besides the area exposed, there is another important factor, viz., the heat penetration into the depth or thickness of the metallic boundary walls, at each filling of steam. In other words, and in most engines, a certain weight of metal, after being previously exposed and cooled to nearly the condenser temperature, is again heated to about that of the steam, at the expense of the heat of the latter. This depth of heat-penetration is found to be much smaller with the hotter than with the colder walls, and with the hotter cylinders there is much less steam condensed. The heat-penetration per revolution is not always understood. Taking the case of a small vertical non-jacketed engine making 50 revolutions per minute, experiments have shown that, at a depth of about 7 millimetres from the inner surface, the

temperature is constant; but at a depth of 1½ and 3 millimetres from the same surface, the mercurial column of the thermometer rises and falls regularly at each revolution—rising with the hot steam, and falling with the cooler exhaust stroke. The heat-range of the metal at 1½ millimetres from the internal surface is much greater than that at 3 millimetres. The greater the depth in the metal, the less this range of heat. The internal surface when dry and clean probably assumes the whole range of temperature, say 100° F. Taking a vertical section of the cylinder 1 millimetre thick, there will be a wedged-shaped part of the thermal gradient of this thickness, varying in temperature per revolution, from about 100° F. at the inner surface to nothing at a depth of 7 millimetres. Similar heat curves, or thermal gradients are shown in Plate 8. By taking the average depth, and the average temperature range of the metal, the weight of metal heated and cooled so many degrees per stroke can be calculated. This should give the number of thermal units lost by the steam, representing so much water per stroke of the piston.

The number of square feet of internal surface, and the weight of steam condensed per hour in the cylinder being known, the number of lbs. of steam per hour per square foot can be calculated for a certain steam-range and metal-temperature. Any device that reduces heat-penetration will certainly tend to increase economy. Increasing the speed has the same effect, by reducing the time of exposure to the cooling influences. In the experiments, it may be seen how much the initial pressures and the areas of the diagrams vary with the different cylinder-temperatures, the hotter walls giving much larger diagrams. The quantity of water produced by the condensation of the steam varies from 1 lb. to 43 lbs. per hour per square foot of internal surface, according to the quality of the steam and temperature of the surfaces. With the Parsons rotary steam-engine there is no such alternate heating and cooling of the metal.

When engineers desire to condense steam, they arrange to keep all the condensing-surfaces much cooler than the steam. In many engine-cylinders this is unconsciously done, in a less degree; much water being produced, because the boundary walls are colder than the steam that touches them. The results of heating the surfaces are shown in these experiments. Superheated steam is used with the same object, as it must become saturated before any condensation takes place. Neither the heated metal wall nor the superheated steam, however, seem to stop condensation entirely, as appears from the diagrams. The presence of water in various

forms in the cylinder, as fog, mist, drops, &c., has probably some effect upon the steam.[1]

It is now generally recognized by steam-engine authorities, that for the maximum economy in any engine, the exhaust should be steam only, and not a mixture of steam and water; on the other hand, it should not be allowed to escape superheated, or again there will be loss. If there is water present at exhaust, its evaporation from the walls carries much heat to waste into the condenser or into the air. If there is no water present in the mixture, the walls are practically dry, and no heat is lost from the walls to the condenser. Professor Dwelshauvers Dery has lately carefully pointed this out,[2] and quotes some of the Author's experiments on this point.

The use of a bad heat-conductor does not, unfortunately, prevent condensation, as will be seen in the experiments with glass walls. Covering the metallic surfaces with varnish has a considerable effect in diminishing condensation, and some experiments on this point are added. The tests show that the temperature of the metallic surfaces has the greatest influence on condensation, the hotter walls reducing it considerably, and at the same time increasing the steam temperature range, so that there is less condensation with the greater ranges.

The Author's experiments had special reference to the following:

1. Variations of the size of the cylinders, and in this way changing the ratio of the volume, to the surface touched by the steam. In two sets of experiments non-heated surfaces were added, until very little steam was left to form a diagram. 2. Varying the range of temperature of the steam in the cylinders by working condensing or non-condensing; the difference between the steam temperature by gauge, immediately before admission, and that at the end of the exhaust stroke, is called the steam temperature range. These temperatures are taken from the corresponding pressures. 3. Working with ordinary or slightly super-heated steam. 4. Varying the material of the cylinders;—cast-iron, phosphor-bronze, or glass. 5. Varying the temperature of the cylinder-walls in the different experiments, so that their internal surfaces were either hotter or colder than the temperature of the steam first touching them, all other conditions being the same. 6. Variations

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[1] As early as 1859, John Penn, then President of the Institution of Mechanical Engineers, very clearly and exactly defined the action of steam in a steam-engine. Proc. Inst. Mech. Engs., 1859.

[2] Étude Calorimetrique de la Machine à Vapeur," Paris, 1892.

of time, and its effect on the pressure of the steam touching the surfaces, as shown by the indicator diagrams. This was done by varying the number of fillings per minute. Steam in contact with hot or cold walls produces marked differences in the condensation during extremely short periods of time. 7. Changing the working pressure of the steam in lbs. per square inch within certain limits, to alter the steam temperature range. 8. Varying the depth of heat-penetration into the metal walls at each steam filling by altering the surface-temperatures.

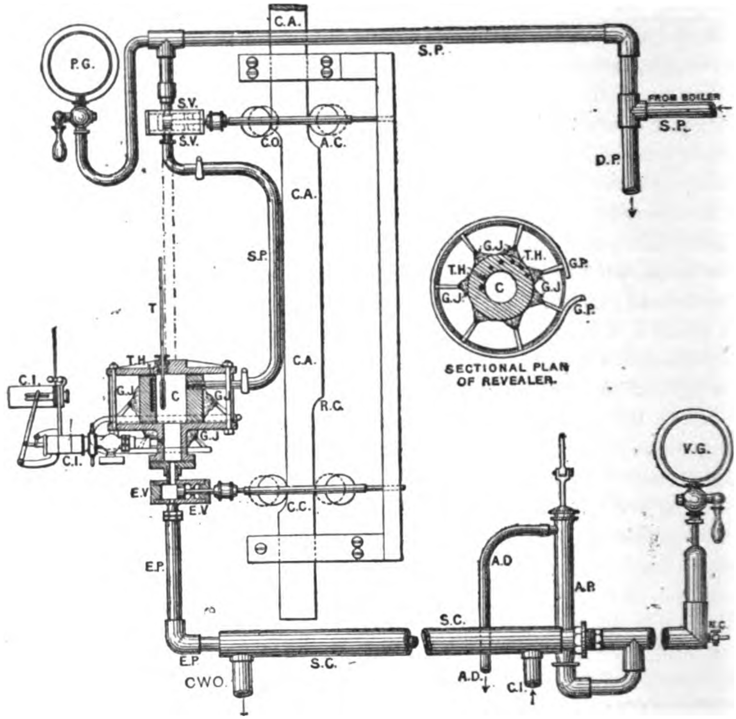
9. Variations in the temperatures of the cylinders throughout their thickness. 10. The area of the indicator diagrams taken in the usual way under the different conditions, and their shape, after steam was cut off. 11. The "dryness fraction" of the steam both at cut-off and release, giving the amount of steam and water present in the mixture in the different experiments. 12. The weight of water condensed per square foot (of internal surface touched by the steam) per hour with different cylinder temperatures, and different steam temperature ranges. Comparison of the amounts of condensation with the steam temperature ranges. 13. The weight of steam used for the different working conditions. 14. Varnishing the surfaces exposed to the steam.

Experimental Apparatus.

Fig. 1 shows a general drawing of the apparatus used, as fixed in the engine-house at Bermondsey, where steam, water, and gas power were available for the purposes of the experiments.

Steam was conveyed from the main steam-pipe by the small well-covered pipe, shown at the top of the drawing, to the cylinder or revealer under test. This pipe was provided with a trap to keep back any water. The steam-gauge was duly tested. The two piston-valves, one at the top for the steam, and another at the bottom for the exhaust, were worked by four cams. These cams and the indicator were driven by a reciprocating lever from the parallel-motion of the steam-engine. The two upper cams actuated the steam-valve, one letting the steam into the cylinder and the other cutting it off at the required moment. The two lower cams moved the exhaust-valve, one opening it at about 95 per cent. of the stroke, and the other shutting it a short time before steam was admitted. The surface-condenser consisted of two thin small brass pipes, one inside the other. The steam thus condensed passed through the small vertical air-pump into a glass receiver, where it was afterwards weighed. The circulating-water taken

Fig. 1.



GENERAL ARRANGEMENT OF THE EXPERIMENTAL APPARATUS.

Index to Parts.

SP	represents	Steam-pipes.	**T**	represents	Thermometer.
DP	„	Drain-pipe.	**TH**	„	Temperature holes.
PG	„	Pressure-gauge.	**CI**	„	Crosby indicator.
SV	„	Steam- (piston) valve.	**EP**	„	Exhaust-pipe.
C	„	Cylinder or revealer.	**SC**	„	Surface Condenser.
CA	„	Cam apparatus.	**CI**	„	Circulating (water) inlet.
AC	„	Admission cam.			
CO	„	Cut-off cam.	**CWO**	„	Circulating (water) outlet.
EV	„	Exhaust- (piston) valve.			
RC	„	Release cam.	**AP**	„	Air-pump.
CC	„	Compression cam.	**AD**	„	Air-pump discharge.
GP	„	Gas-pipe.	**VG**	„	Vacuum-gauge.
GJ	„	Gas-jets.	**NC**	„	Non-condensing cock.

The cam apparatus (CA) is pulled up and down vertically by a lever from an adjacent steam-engine.

from an adjoining water-pipe, after passing through the surface-condenser, was led into a small tank on a weighing-machine. A Crosby indicator was fixed horizontally, having a short and direct connection with the cylinder. Vertical temperature-holes were drilled in the cylinder walls. These were filled with mercury, and into the mercury small 3-millimetre glass mercurial thermometers were inserted. Thin steel temperature-cups were fixed inside the cylinders, as shown in the drawing, exposed all round to the steam. In some of the experiments, the bulb of a thermometer was also placed inside the cylinder. When it was desired to raise the temperature of the cylinder walls, a row of small gas-jets acted on the outside of the cylinder, and another row acted upon the bottom cover. The first set of jets heated the top cover as well as the external walls. The temperature of the cylinders could thus be very readily raised or lowered a few degrees, or kept at a constant temperature.

The surface-condenser was used for the non-condensing as well as for the condensing tests, but in the former a small cock was opened to the atmosphere. In this way the exhaust was at atmospheric pressure—the apparatus working as if non-condensing, the vacuum-gauge was at zero, and all the steam used was condensed and weighed as in the condensing experiments.

Each experiment lasted 40 minutes, and was divided into two distinct tests of 20 minutes each. If the two did not agree closely, they were condemned and repeated. Experiments of much longer duration were also made, but they always gave practically the same results as the shorter ones. Before a test was commenced, the apparatus was worked for about half an hour under the required conditions, so that all temperatures became normal. Indicator diagrams were obtained every 10 minutes, and an average of the four was taken. Those given in Plate 7 are the nearest to the mean. Equal parts along the base lines of the diagrams do not therefore represent equal times. One diagram (Fig. 2, Plate 7) is added representing pressures for equal times. The external portions of the cylinders and ends were left uncovered. The temperatures of the cylinder-walls were noted every 10 minutes, the pressure of steam and the vacuum every 5 minutes. The temperature of the hot circulating-water was recorded every minute, and that of the cold water every 2 minutes. This water was run into a small tank and weighed. The air-pump discharge, giving the quantity of steam used, was weighed, and its temperature taken. A counter gave the number of fillings per minute. The steam- and exhaust-valves and the

surface-condenser were frequently tested and found tight. To determine the exact volume of the different cylinders and pipes, &c., two methods were adopted: they were filled with water and the latter weighed; the volumes were also calculated, and these gave the same results. The exact volumes were necessary in order to obtain the "dryness fraction" at cut-off and at release, &c. A heat-balance was calculated in nearly all the experiments. The number of pounds of steam used multiplied by its total heat gave the number of thermal units per stroke received by the apparatus. The total heat rejected in thermal units per stroke was,—(1) the air-pump discharge in pounds per stroke multiplied by its temperature; (2) the pounds of circulating water per stroke multiplied by its rise in temperature; (3) the radiation in thermal units; and (4) the thermal units per stroke unaccounted for.

The following is a list of the various cylinders experimented on. Three of these are shown in Figs. 3, 4 and 5, Plate 7, and three in *Figs. 3a, 4a,* and *5a*, pp. 288, 289.

No. 1. *Glass Cylinder and Cast-Iron Covers.*—2 inches internal diameter, 3 inches long. Volume 0·00838 cubic foot; surface of glass (vertical walls) 0·13 square foot and 0·39 square foot of metal, a ratio of 1 cubic foot of volume to 62 square feet of surface. Glass walls $\frac{1}{8}$-inch thick, wrought-iron cylinder outside, and cement between. No temperature-holes in the wall. Two experiments, Nos. 13 and 14, condensing and non-condensing, with cold walls only, and saturated steam.

No. 2. *Glass Cylinder Air-Jacketed and Cast-Iron Covers.*—2·7 inches in internal diameter, 3 inches long. Volume 0·0135 cubic foot; surface 0·177 square foot of glass and 0·42 square foot of metal—a ratio of 1 cubic foot of volume to 44 square feet of total surface. Two glass cylinders were used, one within the other, with annular air-jacket between. Inner cylinder, 2·70 inches in internal diameter; outer cylinder, 3·76 inches in internal diameter; both 3 inches long. Thickness of wall of inner glass cylinder 0·145 inch. No temperature-holes. One experiment, No. 23, with cold walls, condensing, saturated steam. These glasses stand the steam-pressure well, if care be taken to heat them very gradually with hot water (*Fig. 3a*, p. 288).

No. 3. *Glass Steam-Jacketed Cylinder and Cast-Iron Covers.*— Two vertical glass cylinders, as before, with annular space and steam between forming a steam jacket. Inner cylinder 2·7 inches in internal diameter, 3 inches long. Volume 0·01378 cubic foot; surface 0·179 square foot of glass, and 0·422 square foot of metal—a ratio of 1 cubic foot of volume 43½ square feet of surface.

Inner glass 0·145 inch thick. One experiment, No. 33, with hot glass walls, saturated steam, condensing (*Fig. 3a*, p. 288).

No. 4. *Phosphor-Bronze Cylinder and Cast-Iron Covers* (Fig. 5, Plate 7).—5½ inches in internal diameter, 3 inches long. Volume 0·04612 cubic foot; surface 0·366 square foot of phosphor-bronze, and 0·612 square foot of iron—a ratio of 1 cubic foot of volume to 21 square feet of surface. Walls of cylinder ½ inch thick, with four holes for taking temperatures. Walls heated externally by gas. Tests with hot and cold walls, saturated and super-heated steam. Six experiments, Nos. 27 to 32, condensing and non-condensing.

No. 5. *Cast-Iron Cylinder and Covers* (Fig. 3, Plate 7).—2 inches in diameter, 3 inches long. Volume 0·009219 cubic foot; surface, 0·62 square foot—a ratio of 1 cubic foot of volume to 67 square feet of surface. Wall 1 inch thick. This thickness was adopted as that generally used in steam-cylinders, but thinner walls were also experimented upon. Eight temperature-holes through the thickness of wall. Tests with hot walls heated by gas-jets, and with cold walls. Experiments made at full and half speeds, and with two different ranges of steam-temperature. Saturated steam. Eight experiments, Nos. 2 to 12, condensing and non-condensing.

No. 6. *Cast-Iron Cylinder and Covers* (Fig. 4, Plate 7).—5½ inches in diameter, 3 inches long. Volume 0·04411 cubic foot; surface 1·28 square foot—a ratio of 1 cubic foot of volume to 29 square feet of surface. Walls ⅝ inch thick, with five temperature-holes. Here the internal surface was much less compared with the volume than in the preceding experiments. Both saturated and super-heated steam used. Eight experiments, Nos. 15 to 22. Cold walls and also cylinder heated by gas. Condensing and non-condensing.

No. 7. *Cast-Iron Cylinder and Covers Varnished.*—5½ inches in diameter, 3 inches long. Dimensions the same as those of *No. 6*, but with the vertical walls and top and bottom covers varnished inside. Three coats of varnish were applied, giving smooth glazed surfaces. The varnish stood the temperature well. Total surface varnished 0·84 square foot. Unvarnished and small pipes 0·49 square foot—a ratio of 1 cubic foot of volume to 30 square feet total surface. One experiment, No. 34. Condensing, cold walls with saturated steam (Fig. 4, Plate 7).

No. 8. *Cast-Iron Cylinder and Covers.*—Dimensions of cylinder, the same as in No. 6, but with 24 and also with 12 wrought-iron plates, 0·094 inch thick, placed radially inside the revealer, to increase the surface. Total internal surface, with 24 plates inside, 3·42 square feet, taking both sides of the plates. Total volume

0·03548 cubic foot—a ratio of 1 cubic foot to 96 square feet (the maximum ratio used). With only 12 plates inside the revealer, the volume was 0·03933 cubic foot, and the surface was 2·37 square feet—a ratio of 60 to 1 (*Fig. 4a*, p. 289).

Three experiments, Nos. 35, 36 and 37, were made, condensing, with cold walls and saturated steam, with the principal internal surfaces varnished and unvarnished, so as to determine the effect of varnishing.

No. 9. *Cast-Iron Cylinder and Covers.*—Dimensions—as in No. 6, with one or two rings of cast-iron ⅝ inch thick placed inside. With the small inner ring, 2¼ inches in internal diameter by 2¾ inches long, surface 1·50 square foot. Volume 0·03577 cubic foot. Ratio 42 to 1. With large inner ring, 3¾ inches in internal diameter by 2¾ inches long, surface 1·77 square foot. Volume 0·03157 cubic foot. Ratio 56 to 1. With both rings together, surface 2·16 square feet. Volume 0·02281 cubic foot. Ratio 95 to 1. (*Fig. 5a*, p. 289.)

Three experiments were made, Nos. 38, 39 and 40, condensing, cold walls, and with saturated steam. No surfaces varnished.

On the Forms Assumed by Water Condensed from Saturated Steam. Pressure about 30 lbs. per Square Inch. Fillings, 34 to 35 per Minute.

In endeavouring to describe what was seen inside a double-glass air-jacketed revealer, the remarks are divided into two portions, the steam-stroke and the exhaust-stroke, the phenomena being different during these two periods. The walls were not heated in any way, but their temperature would be much less than that of the steam.

Steam-Stroke, Vertical Glass Walls.—On the admission of steam, the walls were immediately covered with the finest white mist, as well as all sizes of circular drops, from ½ millimetre to 3 millimetres in diameter. The smaller drops were very near together, and the larger ones further apart. The general aspect (full size) was that shown in *Fig. 7*. During the admission of the steam, and before it was cut off, the drops were in rapid motion on all the boundary walls, urged along by the velocity of the entering steam. The 3-millimetre drops ran down the vertical wet surfaces. They never seemed to exceed about 3½ to 4 millimetres in diameter.

Exhaust-Stroke, Vertical Walls.—The instant the exhaust-valve opened, due to the fall of the pressure, drops of all sizes were in violent ebullition. There was instantaneous disappearance of all

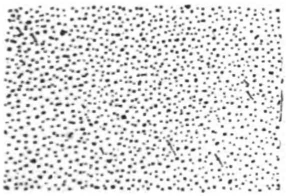
fine mist, the glass surfaces becoming quite clear. All the drops broke up and lost their circular shape. At the end of this stroke, however, many of the larger drops (2 to 3 millimetres in diameter) remained, leaving the walls between quite dry.

Horizontal Cast-Iron Bottom Cover.—All over this bottom cover there was about 1 millimetre to $1\frac{1}{2}$ millimetre of moving water during the steam-stroke. Immediately the exhaust-valve opened, strong ebullition took place during the whole exhaust-stroke. This surface, however, never became quite dry.

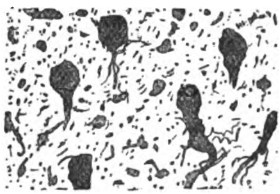
Horizontal Cast-Iron Top Cover.—On this internal surface there were also drops and mist, much as on the vertical walls, which during the admission of steam were also in rapid motion. The drops often fell vertically. During exhaust, the same quick boiling was observed. Working either condensing or non-condensing, the same general phenomena took place, with rather more violent and quicker evaporation during exhaust in the former case.

Fig. 6. *Fig. 7.*

FORMS OF WATER ON ENGINE CYLINDERS.



HOT JACKET WALLS.



COLD NON-JACKETED WALLS.

Other Experiments with Glass and Cast-Iron Walls.—In some other experiments with saturated steam at a pressure of 50 lbs. per square inch, all the phenomena were practically the same, with the addition of a little more moisture, and sometimes films of water on the cold walls. All these forms of moisture were forced by the in-coming steam to move rapidly over the surfaces, splashing in all directions. Sometimes there were no films, but only drops of various sizes. When the glass cylinder was jacketed with steam and its surface was considerably raised in temperature, the wet condition of the glass was very much changed, showing less water present in the mixture, viz., few drops and principally the finest white mist (*Fig. 6*). The above description approximates to a small slow-running single-cylinder engine, and therefore rather more water would be present than in most unjacketed engines.

Double-Glass Revealer fixed on to Steam-Engines.—The Author

has had the opportunity of placing this instrument with an air-jacketed cylinder on various engines, and adds a few notes of the observations made on two of them. The first was fixed on to the low-pressure cylinder of a jacketed compound vertical condensing-engine, indicating some 200 HP., and working at about 60 revolutions per minute. In this cylinder the pressures were under that of the atmosphere. By previous calculations and tests, the steam was known to be practically dry at release, and the revealer confirmed this fact. After the apparatus had been on the cylinder some little time, and the internal parts had assumed their normal temperature, only the finest white mist was seen inside the glass cylinder during the steam-stroke similar to that presented in *Fig. 6*, and disappeared instantaneously at exhaust. There were no large drops like *Fig. 7* and no water films, the internal walls becoming dry before the end of the exhaust. As the revealer was only air-jacketed, the mist or moisture would probably be still less on the actual cylinder walls. With the second engine, and the same air-jacketed revealer, fixed on to the low-pressure cylinder of a compound jacketed condensing-engine, indicating about 50 HP., and running at about 35 revolutions per minute. Here it was also known by experiments that the steam was dry at release. As in the last case, only the finest mist was seen, the particles being perhaps a little larger than before, with rapid disappearance at exhaust. The initial steam-pressure in this cylinder was a little higher than that of the atmosphere.

What has been observed with steam-engines, as to the various shapes the condensed steam assumes on the surfaces during the steam stroke, may be summarized thus: These shapes depend upon the temperature of the internal surfaces, whether they are hotter or colder than the entering steam. In the case of non-jacketed engines, with cylinders much colder than the steam, the drops vary from $\frac{1}{10}$ millimetre to $3\frac{1}{2}$ millimetres in diameter, with the addition of some moving films of water (*Fig. 7*). With efficiently jacketed cylinders, the drops are much smaller in diameter, and probably never exceed $\frac{1}{2}$ millimetre, and there are no films of water (*Fig. 6*). In all cases, there is rapid motion of the various kinds of moisture on all the surfaces and quick evaporation at exhaust. This is with moderate steam-pressures and speeds and fairly dry saturated steam.

From the Author's observations, he considers that the drops and water-films are never at rest in engine-cylinders, either on the vertical or the horizontal surfaces. It is difficult for the eye to follow the storm of mist, fog, drops, films, &c., dashing against the walls in all directions and then evaporating more or less

rapidly at exhaust. With these violent movements inside the cylinders, especially on all the surfaces and passages, during the whole period of the steam-stroke, with the action of the piston wiping off and removing at each stroke all the moisture from the cylindrical portions, it seems natural to conclude that the innermost metallic surfaces should almost immediately assume the varying temperatures of the rapidly-moving steam. The condensed water in all its various wet forms, has probably also an uneconomic effect on the in-coming steam. The action, therefore, not only of the iron, but also of the water, has to be considered. It may be not so much a question of iron *versus* water, but of iron as well as water, that diminishes so considerably economy in steam-engines. With internal-combustion or gas-engines, where the surfaces are very dry, the case is very different. In both, however, there is a certain amount of lubricant which has some effect on all the surfaces upon which condensation can take place. In steam-engines, there is—firstly, a solid and good conductor of heat—cast iron; secondly, a liquid and often bad conductor—water, fog, mist, &c.; and thirdly, a gas—steam, which, if not super-heated, generally contains a small percentage of water.

Limits of Experimental Errors.

As the indicator-springs were checked, the diagrams are as correct as usual, say, within two per cent. As to the weights of feed-water and circulating water, the weighing-machines being verified, the results are doubtless within the same limit of accuracy. The thermometers were also checked, and were found to be practically correct. In taking the various temperatures of the cylinders at different depths, the small vertical temperature-holes were filled with mercury, into which small glass thermometers were placed. Glass being a bad conductor of heat, there is little doubt that in the holes situated nearest to the steam-surfaces, the temperatures of the metal given is less than it would be in reality. The nearer the temperature-holes are to the surface, the greater the temperature range. Fig. 21, Plate 8, shows not only the actual thermal gradient in this manner, but also in dotted lines what the thermal gradient probably would have been, had a perfect thermometer been used. The mean temperatures are no doubt nearly right, as it is only the extent of the ranges that would be influenced. In the non-fluctuating part of the metal of the cylinder, the question is very different, as here the temperatures are constant; they are therefore practically correct.

At some future time, the Author hopes to use either a metallic-bulb thermometer or some type of metallic electrical thermometer or thermopile. The readings of the counter giving the number of fillings per minute are correct. The pressures of steam have often been transposed in these experiments to their corresponding temperatures. This has been done from Professor Dwelshauvers Dery's steam-tables. In order to check the thermometers in position with different pressures of steam, the apparatus was stopped, and steam was allowed to remain for some time inside one of the cylinders. The readings of the glass thermometers were found to correspond with those given by the steam-tables. The steam- and the vacuum-gauges were also often checked and found correct.

Remarks on and Comparisons of Some of the Experiments.

General.—Table I, Appendix, gives the results of the experiments, and the headings of columns 14 and 16 only need explanation. If no steam were condensed, the cylinder would only be required to be filled once; but this is seldom the case. The number of times it was necessary to fill them (up to release) is given in Table I, and varies from 50 to 1. These figures mean, therefore, the number of times it was found necessary to fill each cylinder under test, to make up for the condensation of steam under the different conditions of each experiment. The total volume of steam calculated from the feed-water used, divided by the total volume of the cylinder gives this number. For example, if there was 50 per cent. condensation the cylinder would have to be filled twice. These results can be compared together in so many different ways, that the Author will only make a few comparisons.

Comparing generally the condensing with the non-condensing experiments (having the same temperature of walls) the latter give less condensation of steam per square foot of surface than the former, and the "dryness fraction" is usually higher. The same result takes place when comparing experiments with hot and with cold walls, saturated or super-heated steam. The steam temperature range is less working non-condensing than condensing, and this is probably one reason for the reduced condensation. If, however, the experiments with cold walls, condensing, be compared with hot walls, non-condensing, the steam range is often less, and the condensation less. When heated walls are compared with those not heated, the rates of condensation per square foot of surface exposed, seem to have no relation to the range of the steam-temperature.

Temperature of the Cylinder-Walls.—In reference to the temperature of the cylinder-walls, condensation is always less when they are heated than when not heated, and the "dryness fraction" at release is always higher. The temperature of the metal exposed to the steam seems to have a great effect on the condensation results.

Super-heated Steam.—When super-heated steam is used there is always less condensation than with saturated steam. This holds good whether working condensing or non-condensing, or with heated or un-heated cylinders. It is found also to raise the temperature of the metallic walls.

Different Cylinders.—With regard to the sizes of the cylinders used, the 5½-inch diameter iron cylinder gives much less condensation per square foot of exposed surface, than the smaller 2-inch diameter iron cylinder with the same temperature of walls. With the larger cylinder, the area touched by the steam is much reduced in proportion to the volume.

Heat-Penetration per Stroke.—It will be seen by the drawings of the various thermal gradients on Plate 8, that the heat penetrates into the metal walls at very different depths, not only inversely as their temperatures, but (by working the apparatus faster or slower) in proportion to the time allowed for such penetration. For the same number of fillings, this depth is much greater with cold than with heated walls. For the same temperature of wall, by halving the number of fillings per minute, or in other words doubling the time for the heat to take effect, the depth is considerably increased. *Fig. 8* gives a graphic representation of this heat-penetration per stroke on a temperature base deduced not only from these experiments, with No. 6 cylinder, Fig. 4, Plate 7, but also from some on an experimental steam-engine. The heat-penetration per stroke with hot walls and saturated steam is 1¾ millimetre. The maximum penetration, with a cold wall and at the slowest speed, is 14 millimetres. Less heat-penetration always corresponds with less condensation per square foot. It will be seen that as the walls are raised in temperature so the heat-penetration is reduced.

Plotted Results.—Some of the experiments on Revealer No. 6 have been plotted on a base line of the temperature of the cylinders, as it is this temperature which seems to have the greatest influence in diminishing or increasing condensation. With walls colder than the steam, there is greater condensation per square foot, and with heated walls much less. *Fig. 8* gives:—

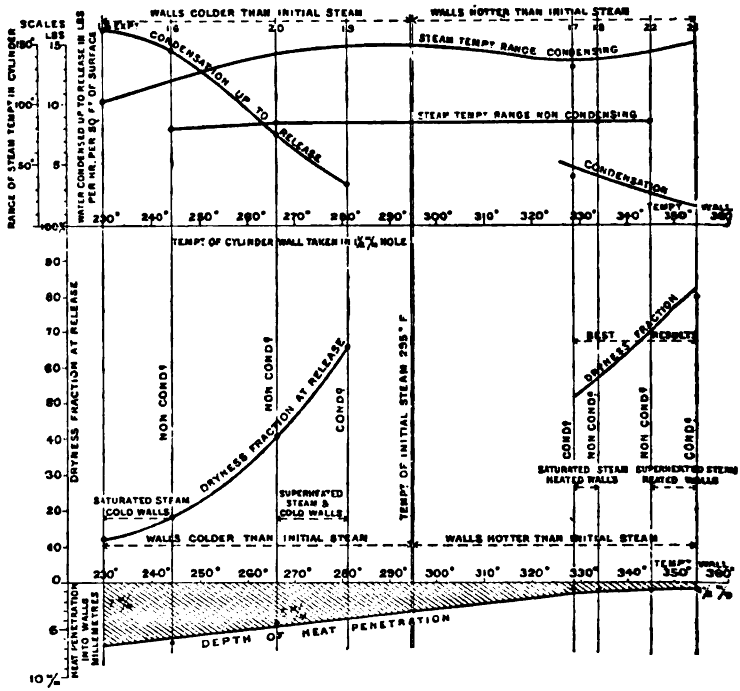
First, the weight of water condensed per hour per square foot of internal surface; secondly, the depths of heat-penetration into

the cylinder-walls; thirdly, the ranges of steam temperature; fourthly, the percentage of steam and water present at release.

It will be seen how the curves of condensation, heat-penetration and water present, all gradually diminish with the increasing temperatures of the cylinders. The steam-range curve does not seem to follow any of the others.

Effects of Time on Condensation.—Comparing two sets of experiments (2 and 11 with the 2-inch cast-iron revealer, Table I),

Fig. 8.



GRAPHIC REPRESENTATION OF STEAM-RANGE; STEAM AND WATER PRESENT, AND HEAT-PENETRATION ON A BASE OF TEMPERATURE OF WALLS.

where the number of fillings per minute was varied considerably, the following results were obtained. With about the same steam temperature range and with cold walls, the condensation is about 40 per cent. less per square foot at half speed than at full speed, whether condensing or non-condensing. So that doubling the time of exposure with the same steam-pressure reduces the condensation considerably. This tends to prove that most of the condensation takes place very quickly.

Condensation Results.—In Table II will be found the various experiments arranged in order, on a scale of pounds of steam condensed per square foot-hour; the different results being arranged according to the amount of condensation. At the top are seen the results with the hot cylinders and super-heated steam, then those of the cooler metal with saturated steam, and at the bottom the maximum condensation with the smaller cylinders and the colder walls.

Comparing experiments 28 and 30 with the same steam range and with saturated steam, the condensation per square foot is about three times greater with cold than with hot walls. With experiments 31 and 32, with the same range and with super-heated steam, the condensation is also about three times greater with the cooler walls. This is repeated in experiments 16 and 18.

Glass Cylinders.—The air-jacket round the glass cylinder is found to reduce condensation and to increase the area of the diagram. With steam in the jacket, there is a gain of 17 per cent. in diminished condensation over air in the jacket. Comparing the glass cylinder with and without steam in the jacket, there is 30 per cent. less condensation, and 83 per cent. more steam present with the same steam range. The cylinders, however, were not of the same diameter. Cold vertical glass walls, condensing or non-condensing, give 22 to 28 per cent. less condensation than the cold metal vertical walls. With less steam range there is less condensation. In many of these experiments the steam range is about the same, but the condensation results vary considerably.

Phosphor-Bronze Cylinder (Vertical walls only).—The comparison of experiments 27 and 28 shows a gain in working non-condensing over working condensing, both with cold walls and with saturated steam, of 12 per cent. reduced condensation with 25 per cent. less range and 6 per cent. more steam present at release. The comparison of experiments 27 and 29, working condensing and with saturated steam, shows a gain of the hot over the cold walls of 63 per cent. reduced condensation, 35 per cent. more range, 85 per cent. less penetration, 119 per cent. more area diagram, and 324·7 per cent. more steam present at release. Here there is less condensation with greater range.

The comparison of experiments 27 and 31, working condensing and with cold walls, shows a gain, by super-heated steam, over saturated, of 59 per cent. reduced condensation, 12 per cent. more range, 85 per cent. less penetration, 76 per cent. more diagram area, and 292 per cent. more steam present at release. Experiments 27 and 32 are both condensing; one with cold walls and

saturated steam, and the other with heated walls and super-heated steam. Here the gain is 88 per cent. less condensation, 16 per cent. more steam range, 109 per cent. more area of diagram, and 521 per cent. more steam present at release. The minimum weight of steam condensed per square foot with heated walls and super-heated steam is on this Table.

In experiments 29 and 32, the quality of the steam only was changed to see the advantage of super-heated over saturated steam, working condensing and with heated walls. The gain was :—68 per cent. less condensation, 15 per cent. less range, 4 per cent. less area diagram, and 46 per cent. more steam present at release. Experiments 28 and 30 are both non-condensing with saturated steam, but one with heated, and the other with cold walls. The gain by the heated walls was 66 per cent. less condensation with the same steam range, 69 per cent. more area of diagram, and 289 per cent. more steam present at release. In this Table the rates of condensation are not at all proportional to the steam ranges; and the best results are with super-heated steam and heated walls.

Cast-Iron Small Cylinder.—The first two experiments, both with cold walls, show that there is 23 per cent. less condensation when working non-condensing than working condensing. There is less condensation with less range.

In comparing experiment No. 2 with Nos. 5 and 6 experiments, both condensing, one with hot and the other cold walls, there is more range with much less condensation. With the hotter walls there is 80 per cent. less heat-penetration, 68 per cent. less condensation, 57 per cent. more area diagram, 650 per cent. more steam present.

With 5½-inch Cast-Iron Cylinder.[1]—In the first pair of experiments (15 and 16), the condensation is 12 per cent. less non-condensing than condensing, with saturated steam and cold walls. In the second pair (15 and 17), only the temperature of the walls was changed; both with saturated steam, condensing. The hotter walls gave 69 per cent. less condensation, 80 per cent. less penetration, and 91 per cent. more area of diagram with 375 per cent. more steam present. Third pair (15 and 19), both condensing, with cold walls. Working with super-heated gave the following advantage over saturated steam: 76 per cent. less condensation, 92 per cent. more area of diagram, 440 per cent. more steam present. Fourth pair (15 and 21), with cold walls and saturated steam against

[1] Compare columns 1, 2, 11, 12, 15, 24, 26 and 30 of Table I for each of the experiments mentioned under this heading.

hot walls and super-heated steam. The advantages in favour of the latter were :—82 per cent. less condensation, 78 per cent. less penetration, 114 per cent. more area, and 500 per cent. more steam present at release. Here the relative amounts of condensation were 16 lbs. and 3 lbs. Cold walls and saturated steam give, therefore, five and a half times more condensation per square foot per hour than with hot walls and super-heated steam. The minimum condensation on this Table is 1·3 lbs. per square foot. The fifth pair of experiments (17 and 21) were with hot walls, condensing. There is a gain of 43 per cent. less condensation, and 27 per cent. more steam present at release with super-heated than with saturated steam. The steam ranges and rates of condensation do not seem to have any relation to each other on this Table.

Comparison of Iron and Phosphor-Bronze Cylinders of same size.[1] —First pair of experiments (15 and 27) with cold walls, condensing, saturated steam. The disadvantage of phosphor-bronze is 47 per cent. more condensation, and 43 per cent. more heat-penetration. Second pair (17 and 29) with hot walls, condensing, saturated steam. Phosphor-bronze gives 79 per cent. more condensation with about same heat-penetration and the same area of diagram. Third pair (19 and 31) with cold walls, condensing, super-heated steam. Phosphor-bronze gives 160 per cent. more condensation and 70 per cent. less heat-penetration.

Fourth pair (21 and 32) with hot walls, condensing, super-heated steam. In this case only is there a slight gain with the phosphor-bronze super-heated steam and hot walls, viz., about 2 per cent. less condensation. In these tests the various steam ranges were not proportional to the rates of condensation. In some cases there is less condensation with greater range, and in others more condensation with less range. Cast-iron seems to be more suitable than phosphor-bronze for a cylinder metal.

Effect of Varnishing the Internal Iron Surfaces.[2]—The first pair of experiments (15 and 34) is with iron cylinders, the chief surfaces being varnished, cold walls, condensing, saturated steam. There is 19 per cent. less condensation per square foot due to the varnishing. Second pair (36 and 37); same cylinder, with the addition of thin plates, increasing the surface considerably. There is a gain of 6 per cent. less condensation due to the varnishing.

Increased area of the Surfaces exposed to Steam.—Experiments

[1] Compare columns 1, 2, 11, 12, 15, 24, 26, 30 and 31 of Table I for each pair of experiments mentioned.

[2] Compare columns 1, 2, 11, 12, 15, 24, 26, 30 and 31 of Table I for each of the mentioned pair of experiments.

15 and 36. The result of doubling the area of the cold surface gives 38 per cent. less condensation per square foot exposed, condensing, saturated steam. In next pair of experiments, Nos. 15 and 35, with 160 per cent. more surface, there is 55 per cent. less condensation per square foot. Thus considerable additions of surface reduce the rate of condensation per square foot; the reason probably being that, as the condensation is so instantaneous on the increased surface, there is less steam left to be condensed. The difference of temperature between the steam and the metal is also much reduced. The steam-pipe and valve were perhaps rather small for the increased quantity of steam. In comparing the three pairs of experiments (15 and 39, 15 and 38, 15 and 40) with cold walls, condensing, saturated steam, and with additional exposed surfaces, consisting of cast-iron rings (instead of thin plates as in the former case), the same general result appears, viz., that the condensation per square foot diminishes with the increased surface, and probably for the same reason. The area of indicator diagrams and heat-penetration also decreases with the increased surfaces. This method of increasing the surface exposed to the steam, approximates to increasing clearance surfaces in steam-engines. For practical application, the above experiments point decidedly to the following requirements for the most economical working. To keep all the cylinder-walls and passages at a somewhat higher temperature than that of the steam in contact with them, by efficient steam-jackets or other means. To diminish as far as possible the areas exposed to such steam in the clearance-surfaces and passages, so as to obtain the maximum volume with minimum surface. To work with as dry a steam as practicable, and to use super-heated steam if possible. To reduce radiation to a minimum. To reduce heat-penetration in the metallic walls. To exhaust the steam from the last cylinder practically dry, so as to contain the minimum quantity of water and heat.

There are not many direct tests on this subject. The Author is acquainted with those of Mr. Escher,[1] made in Switzerland in 1881, and with the interesting experiments of Lieut.-Colonel English, R.E., on condensation in cylinders, described before the Institution of Mechanical Engineers in 1889. These experiments were also at constant volume, but the temperature of the cylinder was not taken. Colonel English is of opinion that condensation of steam is practically instantaneous in steam-engines. With cold walls this is no doubt the case. When, however,

[1] Der Civilingenieur, 1881.

they are heated, *e.g.*, with a steam-jacket, there will be less difference between the temperature of the metal and that of the steam; but, although the interchange of heat may be as rapid, the amount of dew, drops and moisture will be smaller. It is interesting to note that many of these experiments confirm the opinions expressed by Professor Cotterill, F.R.S., in his work on the Steam-engine.

Thermal Diagrams.—Since the above was written, the bulb of a specially sensitive mercurial thermometer has been fixed inside one of the cylinders, and exposed to the steam and the exhaust. The mercurial column rises and falls regularly at each stroke.

The maximum and minimum temperatures can easily be read, and these were noted during each stroke, the usual indicator diagram being taken at a speed of about eleven fillings per minute. At this speed, this thermometer had time to take the whole temperature range, viz., 95° F., corresponding with the maximum and minimum pressures of the indicator diagrams. At 35 revolutions, it followed only about two-thirds of the steam temperature range. The Author has already obtained some preliminary thermal photographic diagrams, and hopes shortly to improve the apparatus and to take others. A comparison could then be made between the usual pressure diagrams, and these thermal diagrams, both being taken at the same time.

The photographic paper is driven to and fro like that for an indicator, and the mercurial column rises and falls. A heat-diagram is thus obtained similar in form to an indicator diagram of pressures, the base-line being in proportion to the stroke, and the verticals in proportion to the varying temperatures.

Drops of water can be seen falling off the bulb of the thermometer, and violent motion inside the glass cylinder, but this rapid movement of the particles of water, on the bulb and elsewhere, does not seem to prevent the quick alternate action of each filling by heating and cooling the mercury. The Author is aware that further improvements can be made in the apparatus, but the above experiments and results are given in the hope that others may be undertaken on a more complete scale and at greater speeds and steam-pressures. He is much indebted to the care and attention of Mr. Golding and others who have assisted him in these investigations.

The Paper is accompanied by a large number of diagrams from a selection of which Plates 7 and 8 and the *Figs.* in the text have been prepared.

APPEN

TABLE I.—EXPERIMENTS ON THE CONDENSATION OF STEAM, IN CYLINDERS OF GLASS, WALL-TEMPERATURES, VOLUMES, AND

One valve for inlet and one for outlet of steam. Different temperatures of wall, con-
steam-filling. 4–8 indicator diagrams taken in each experiment. The minimum depth of
steam, about 50 lbs. Number of fillings and emptyings per minute, about 34–35, sometimes half
Experiments with saturated and super-heated steam. External portion of revealers not covered,
75° and 90° Fahrenheit.

No. of Revealer or Cylinder.	1 Experi-ment No.	2 Conditions of Experiment.	3 No. of Fillings and Emptyings per Minute.	4 Steam-Pressure in Pipe near Revealer. Lbs. Gauge.	5 Lbs. Abso-lute.	6 Corre-sponding Tempera-ture, ° Fahren-heit.
Experiments on 2-inch Glass Revealer (with Cement and Wrought-Iron outside Glass). 2-inch inch thick; and Wrought-Iron, 0·203 inch						
1	13 14	Saturated steam, cold walls condensing non-condensing	34·57 34·92	47·2 48·0	62·0 62·8	295 296
Experiment on 2¾-inch Double-Glass Revealer. Inner Glass Cylinder, 2·70 inches internal diameter, 0·26 inch thick, 2·95 inches						
2	23	Saturated steam, air jacket . . cold walls condensing	34·85	48·5	63·3	296
Experiment on 2¾-inch Double-Glass Revealer with Steam-Jacket. Size of Glasses, about the						
3	33	Saturated steam, jacketed . condensing . . .	34·25	35·0	49·6	280
Experiments on 5½-inch Phosphor-Bronze Revealer. ½-inch Wall, 3 inches long. Top and						
4	27	Saturated steam, cold walls condensing	34·65	49·0	63·8	296
	28	Saturated steam, cold walls non-condensing	34·32	49·0	63·8	296
	29	Saturated steam, hot walls condensing	34·50	49·0	63·8	296
	30	Saturated steam, hot walls non-condensing	35·20	49·0	63·8	296
	31	Steam super-heated about 40° Fahrenheit, cold walls condensing	34·80	49·0	63·8	296
	32	Steam super-heated about 40° Fahrenheit . . hot walls condensing	33·90	49·0	63·8	296

[1] If no steam is condensed, each cylinder would require filling once. These

[Continued on page 286.]

DIX.

PHOSPHOR-BRONZE, AND CAST-IRON, WITH VARIATIONS IN THE TIME OF FILLING,
SURFACES, WITHOUT PISTON, 1892.

densing and non-condensing. Thermal gradients of wall. Cut-off about $\frac{1}{7}$ to $\frac{1}{5}$ in time of heat-penetration per stroke given in millimetres. Total weight of steam used. Pressure of this. Dryness fraction at cut-off and at release. Range of temperature in revealer. except with the glass cylinders. The temperature of the engine-house varied between

7	8	9	10	11	12	13	14	15	16	17
Cut-off taken from Indicator Diagrams Per Cent.	Vacuum in Condenser, Inches Mercury.	Weight of Steam Used.		Wall Action, or depth of Metal at which the Range of Temperature ceases per Stroke in Millimetres.	Indicator Diagram. Mean Pressure, Lbs. per Square Inch.	Steam present in Revealer, as shown by Indicator Diagram in Per Cent. of Total Feed-Water used, or Dryness Fraction of Steam.				
						At Cut-off.		At Release.		Per cent. Difference between Steam present at Release and at Cut-off.
		Lbs. per Hour.	Lbs. per Filling.			Per Cent. Steam.	Fillings of Steam per Stroke.[1]	Per Cent. Steam.	Fillings of Steam per Stroke.[1]	
internal diameter, 3 inches long. Glass Vertical Wall, 0·18 inch thick. Cement, 0·133 thick. Top and Bottom Covers of Brass.										
14·3	25	18·31	0·00882	not	27·61	10·7	9·31	4·2	23·5	6·5
14·4	..	13·21	0·00630	taken	23·68	12·1	8·26	4·7	21·2	7·4
internal diameter, 0·145 inch thick, 3 inches long. Outer Cylinder, 3·76 inches long. Air in Annular Space between Glasses.										
17·4	24	17·46	0·00835	not taken	32·26	17·6	5·69	9·0	11·1	8·6
same as in previous Experiment. 1-inch Felt round Outer Glass and Top and Bottom Covers.										
12·3	24	including 15·51	jacket-water 0·00755	not taken	23·66	17·4	5·74	8·7	11·5	8·7
Bottom Covers of Cast-Iron. See Figs. 9, 10, 11, 12, Plate 7, for Mean Indicator Diagrams.										
14·7	19	27·00	0·01299	10	20·26	25·1	3·99	12·3	8·16	12·8
15·6	..	23·97	0·01164	10	20·41	27·7	3·61	13·0	7·71	14·7
15·9	23	18·66	0·00877	1½	44·29	65·0	1·54	52·2	1·92	12·8
16·9	..	14·34	0·00679	1½	34·59	64·4	1·55	50·6	1·98	13·8
15·8	20	17·01	0·00815	1½	36·00	58·6	1·71	48·2	2·07	10·4
17·7	18	12·07	0·00593	1½	42·42	90·8	1·10	76·4	1·31	14·4

columns represent the actual number of fillings required, calculated from feed-water.

[Continued on page 287.]

TABLE I.—

No. of Revealer or Cylinder.	1	2			3	4	5	6
	Experiment No.	Conditions of Experiment.			No. of Fillings and Emptyings per Minute.	Steam-Pressure in Pipe near Revealer. Lbs. Gauge.	Lbs. Absolute.	Corresponding Temperature, ° Fahrenheit.
Experiments on 2-inch Cast-Iron Revealer, 2 inches internal diameter, 1-inch Wall, 3 inches also Figs. 21, 22, 23, 24, 25,								
5	2	Saturated steam, full speed, and full range of temperature .	cold walls	condensing	34·17	47·1	61·7	294
	mean of 3 and 4		cold walls	non-condensing	33·63	47·0	61·7	294
	mean of 5 and 6		hot walls	condensing	34·28	49·5	64·4	297
	mean of 7 and 8		hot walls	non-condensing	34·67	49·3	64·2	297
	9	Half range of temperature in revealer, saturated steam . . .	cold walls	condensing	34·50	low steam 6·8	21·6	232
	10		cold walls	non-condensing	34·17	low steam 17·5	32·3	254
	11	Half speed, full steam, saturated. . . .	cold walls	condensing	half speed 17·45	48·0	62·9	296
	12		cold walls	non-condensing	half speed 16·10	48·0	62·9	296
Experiments on 5½-inch Cast-Iron Revealer; ⅝-inch Walls, 3 inches long. All Metal also Figs. 29, 30, 31, 32,								
6	15	With ordinary saturated steam	cold walls	condensing	34·70	46·9	61·7	294
	16		cold walls	non-condensing	34·70	47·3	62·1	295
	17		hot walls	condensing	35·37	48·3	63·2	296
	18		hot walls	non-condensing	35·30	48·4	63·4	296
	19	With steam superheated about 40° Fahrenheit	cold walls	condensing	34·25	47·8	62·3	corr. temp. of saturated steam. 295
	20		cold walls	non-condensing	34·50	48·5	63·1	296
	21		hot walls	condensing	34·67	48·1	62·9	296
	22		hot walls	non-condensing	34·17	49·0	63·8	296
Experiments on 5½-inch Cast-Iron Revealer as above, with								
7	34	Saturated steam, internal surface varnished . .	cold walls	condensing	34·85	48·4	63·1	296

[*Continued on page* 288.]

ontinued.

7	8	9	10	11	12	13	14	15	16	17
Cut-off taken from Indicator Diagrams per Cent.	Vacuum in Condenser, Inches Mercury.	Weight of Steam Used.		Wall Action, or depth of Metal at which the Range of Temperature ceases per Stroke in Millimetres.	Indicator Diagram. Mean Pressure, Lbs. per Square Inch.	Steam present in Revealer, as shown by Indicator Diagram in Per Cent. of Total Feed-Water used, or Dryness Fraction of Steam.				
						At Cut-off.		At Release.		Per cent. Difference between Steam present at Release and at Cut-off.
		Lbs. per Hour.	Lbs. per Filling.			Per Cent. Steam.	Fillings of Steam per Stroke.	Per Cent. Steam.	Fillings of Steam per Stroke.	
long. All Metal Walls. See Figs. 13, 14, 15 *and* 16, *Plate* 7, *for Mean Indicator Diagrams;* 26, 27 *and* 28, *Plate* 8.										
17·5	25	27·50	0·01341	9	22·40	7·0	14·2	2·2	44·7	4·8
13·7	..	21·18	0·01050	8	17·57	7·7	12·9	1·9	51·5	5·8
15·2	26	10·28	0·00500	1¾	42·98	24·7	4·05	16·5	6·06	8·2
15·3	..	11·53	0·00555	1¾	26·69	16·9	5·94	6·6	15·4	10·3
5·6	23	9·29	0·00449	9	5·025	5·7	17·4	2·0	49·4	3·7
13·6	..	6·45	0·00315	9	7·575	9·4	10·6	3·8	25·9	5·6
15·5	20	15·98	0·01527	14	27·12	6·8	14·6	2·4	41·8	4·4
15·4	..	12·69	0·01314	14	23·46	6·7	15·0	2·3	43·2	4·4
Walls. See Figs. 17, 18, 19 *and* 20, *Plate* 7, *for Mean Indicator Diagrams;* 33, 34, 35 *and* 36, *Plate* 8.										
17·6	22	24·05	0·01155	7	22·44	28·5	3·51	12·1	8·24	16·4
17·4	..	21·87	0·01050	7	23·40	31·6	3·16	14·4	6·92	17·2
17·0	21	15·35	0·00723	1½	42·84	72·2	1·38	57·5	1·74	14·7
15·7	..	12·65	0·00598	1½	33·36	69·9	1·43	49·5	2·02	20·4
						assuming weight of super-heated steam same as that of saturated.				
16·9	25	14·23	0·00693	5	43·05	75·4	1·33	65·6	1·52	9·8
17·1	..	15·54	0·00750	5	28·42	48·2	2·07	37·2	2·69	11·0
18·3	23	14·06	0·00676	1½	48·03	84·2	1·19	72·8	1·37	11·4
18·7	..	9·75	0·00475	1½	38·46	94·3	1·06	82·0	1·22	12·3
Inner Wall and Top and Bottom Covers Varnished Inside.										
13·1	24	21·33	0·0102	5	23·25	39·5	2·53	16·9	5·92	22·6

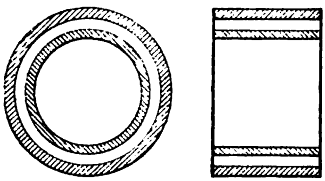
[*Continued on page* 289.]

TABLE I.—

No. of Revealer or Cylinder.	1	2	3	4	5	6
	Experiment No.	Conditions of Experiment.	No. of Fillings and Emptyings per Minute.	Steam-Pressure in Pipe near Revealer.		
				Lbs. Gauge.	Lbs. Absolute.	Corresponding Temperature, ° Fahrenheit.
Experiments on same 5½-inch Cast-Iron Revealer, with 24 and 12 Radial Plates, both Varnished and						
8	35	Saturated steam; Experiment with 24 radial plates not varnished; cold walls; condensing	34·57	48·7	63·4	296
8	36	Saturated steam; Experiment with 12 radial plates not varnished; cold walls; condensing	34·40	48·5	63·4	296
8	37	Saturated steam; Experiment with 12 radial plates varnished . . ; cold walls; condensing	34·90	48·9	63·5	296
Experiments on same 5½-inch Cast-Iron Revealer, with 1 or 2 Inner Rings of Internal Surface						
9	38	Saturated steam; With large inner ring . . .; cold walls; condensing	34·87	48·7	63·2	296
9	39	Saturated steam; With small inner ring . . .; cold walls; condensing	34·75	49·1	63·6	296
9	40	Saturated steam; With both rings .; cold walls; condensing	34·17	49·2	63·8	296

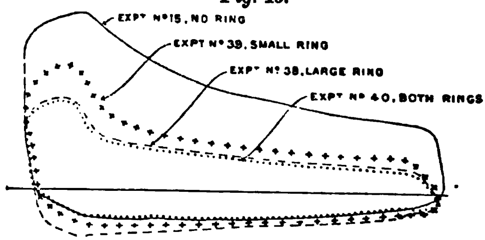
[*Continued on page* 290.]

Figs. 3a.



REVEALERS NOS. 2 AND 3, GLASS.
Diameters, 2¾ inches and 4 inches.

Fig. 4b.

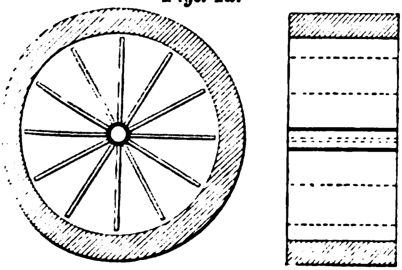


continued.

7	8	9	10	11	12	13	14	15	16	17
Cut-off taken from Indicator Diagrams Per Cent.	Vacuum in Condenser, Inches Mercury.	Weight of Steam Used.		Wall Action, or depth of Metal at which the Range of Temperature ceases per Stroke in Millimetres.	Indicator Diagram. Mean Pressure, Lbs. per Square Inch.	Steam present in Revealer, as shown by Indicator Diagram in Per Cent. of Total Feed-Water used, or Dryness Fraction of Steam.				
						At Cut-off.		At Release.		Per cent. Difference between Steam present at Release and at Cut-off.
		Lbs. per Hour.	Lbs. per Filling.			Per Cent. Steam.	Fillings of Steam per Stroke.	Per Cent. Steam.	Fillings of Steam per Stroke.	
$\frac{1}{16}$ inch thick inside Revealer. Internal Surface of Revealer, Covers and Plates not Varnished.										
11·3	23	26·39	0·01272	6	5·76	5·7	17·6	4·1	24·3	1·6
10·7	22	25·99	0·01259	9	9·24	10·9	9·20	6·5	15·3	4·4
11·9	23	24·97	0·01192	8	14·91	18·4	5·43	8·9	11·2	9·5
Cast-Iron $\frac{5}{8}$ inch thick inside Revealer. not Varnished.										
11·8	24	24·89	0·01192	4	11·745	12·2	8·16	6·0	16·5	6·2
13·5	24	24·42	0·01171	5	13·69	16·8	5·95	6·3	15·8	10·5
13·0	23	25·80	0·01258	2	8·925	7·0	14·2	2·3	43·5	4·7

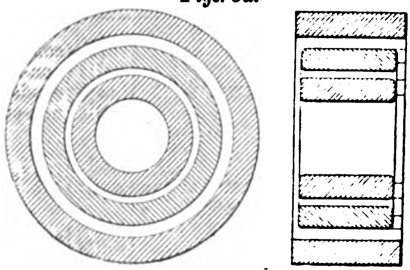
[Continued on page 291.]

Figs. 4a.



REVEALER No. 8, CAST-IRON.
Diameter. 5¼ inches.

Figs. 5a.



REVEALER No. 9, CAST-IRON.
Diameter, 5¼ inches.

TABLE I.—

No. of Revealer or Cylinder.	1	2	18	19	20	21
	Experiment No.	Conditions of Experiments.	Pressures and Corresponding Temperatures as taken from Indicator			
			Initial pressure.		Release Pressure.	
			Lbs. Absolute.	Degrees Fahrenheit.	Lbs. Absolute.	Degrees Fahrenheit.
Experiments on 2-inch Glass Revealer (with Cement and Wrought-Iron outside Glass. 2-inch inch thick; and Wrought-Iron 0·203 inch						
1	13 14	Saturated steam, cold walls condensing non-condensing	57·2 57·9	289 290	27·2 33·3	245 256
Experiment on 2¾-inch Double-Glass Revealer. Inner Glass Cylinder, 2·70 inches internal diameter, 0·26 inch thick, 2·95 inches						
2	23	Saturated steam, air jacket . . cold walls condensing	54·7	286	31·2	253
Experiment on 2¾-inch Double Glass Revealer with Steam Jacket. Size of glasses about the						
3	33	Saturated steam, steam jacketed . . . . condensing	43·7	272	24·2	238
Experiments of 5½-inch Phosphor-Bronze Revealer. ½-inch Wall, 3 inches long. Top and Diagrams and						
4	27	Saturated steam, cold walls condensing	41·8	270	26·2	242
	28	Saturated steam, cold walls non-condensing	48·2	279	31·9	254
	29	Saturated steam, hot walls condensing	57·1	289	46·2	276
	30	Saturated steam, hot walls non-condensing	56·7	289	47·6	278
	31	Superheated steam about 40° Fahrenheit . . . cold walls condensing	50·2	281	42·1	270
	32	Superheated steam about 40° Fahrenheit . . . hot walls condensing	55·6	288	47·3	277

[*Continued on page* 292.]

continued.

22	23	24	25	26	27	28	29	30	31
in Revealer, Diagrams.		Temperature in Steam Pipe Minus Final Temperature= Range of Temperature in Revealer. ° Fahr.	Actual Temperature (taken by Thermometer).			Total Volume of Steam in Revealer and Total Surface (internal) exposed to same, per Stroke.			Steam Condensed in Revealer up to Release in lbs. per Hour per Square Foot of Internal Surface. Lbs.
Final Pressure at end of Exhaust.			Temperature of Wall in 1½-Millimetre hole.		Average Temperature of whole Wall. ° Fahr.				
Lbs. Absolute.	Degrees Fahrenheit.		Average Temperature. ° Fahr.	Hotter or Colder than Initial temperature. ° Fahr.		Volume in Cubic Feet.	Surface in Square Feet.	Ratio = Surface/Volume.	
internal diameter, 3 *inches long. Glass Vertical Wall,* 0·18 *inch thick. Cement,* 0·133 *thick. Top and Bottom Covers of Brass.*									
4·5	158	136·6	not	tak	en	0·008381	0·13 glass; 0·39 iron, &c.; 0·52 total	sq. ft. cub. ft. 62·0 to 1	33·7
15·0	213	82·3	not	tak	en				24·2
internal diameter, 0·145 *inch thick,* 3 *inches long. Outer Cylinder,* 3·76 *inches long. Air in Annular Space between Glasses.*									
4·75	160	135·8	not	tak	en	0·0135	0·177 glass; 0·420 iron, &c.; 0·597 total	44·2 to 1	26·6
same as in previous Experiment. 1-*inch Felt round Outer Glass and Top and Bottom Covers.*									
3·9	152	128·4	not	tak	en	0·01378	0·179 glass; 0·422 metal; 0·601 total	43·6 to 1	22·2
Bottom Covers of Cast-Iron. See Figs. 9, 10, 11 *and* 12, *Plate* 7, *for Mean Indicator Drawing of Revealer.*									
8·6	186	110·0	227	69 colder	226	0·04612	0·366 Ph. B.; 0·612 iron; 0·978 total	21·2 to 1	24·2
15·4	214	82·2	238	58 colder	238				21·3
3·3	146	150·8	334	37 hotter	340				9·12
15·0	213	83·3	335	39 hotter	347				7·25
6·5	174	123·0	273	assuming temp. of saturated steam. 24 colder	273				9·91
5·5	168	128·2	341	44 hotter	343				2·91

[*Continued on page* 293.]

TABLE I.—

No. of Revealer or Cylinder.	1	2	18	19	20	21
			Pressures and Corresponding Temperatures as taken from Indicator			
			Initial Pressure.		Release Pressure.	
	Experiment No.	Conditions of Experiments.	Lbs. Absolute.	Degrees Fahrenheit.	Lbs. Absolute.	Degrees Fahrenheit.
Experiments on 2-inch Cast-Iron Revealer, 2 inches internal diameter, 1 inch Wall, 3 inches		*Drawing of Revealer. See also Figs. 21, 22, 23, 24,*				
5	2	Saturated steam full speed, and full range of temperature . cold walls condensing	50·8	282	20·5	229
5	mean of 3 and 4	Saturated steam full speed, and full range of temperature . cold walls non-condensing	54·5	286	25·7	241
5	mean of 5 and 6	Saturated steam full speed, and full range of temperature . hot walls condensing	61·4	294	41·6	269
5	mean of 7 and 8	Saturated steam full speed, and full range of temperature . hot walls non-condensing	60·4	293	33·0	256
5	9	Half range of temperature in revealer, saturated steam . . . cold walls condensing	17·1	220	9·7	192
5	10	Half range of temperature in revealer, saturated steam . . . cold walls non-condensing	29·5	249	21·4	231
5	11	Half speed, full steam, saturated steam . . . cold walls condensing	58·2	290	25·7	241
5	12	Half speed, full steam, saturated steam . . . cold walls non-condensing	58·3	291	31·0	252
Experiments on 5½-inch Cast-Iron Revealer; ⅝-inch Walls, 3 inches long. All Metal		*Drawing of Revealer. See also Figs. 29, 30, 31, 32, 33,*				
6	15	With ordinary saturated steam cold walls condensing	45·6	275	26·9	244
6	16	With ordinary saturated steam cold walls non-condensing	51·1	282	33·1	256
6	17	With ordinary saturated steam hot walls condensing	56·7	289	46·1	276
6	18	With ordinary saturated steam hot walls non-condensing	57·2	289	44·9	274
6	19	With steam superheated by gas about 40° Fahrenheit . . . cold walls condensing	52·1	283	45·3	275
6	20	With steam superheated by gas about 40° Fahrenheit . . . cold walls non-condensing	49·6	280	41·4	269
6	21	With steam superheated by gas about 40° Fahrenheit . . . hot walls condensing	58·3	291	50·6	282
6	22	With steam superheated by gas about 40° Fahrenheit . . . hot walls non-condensing	58·7	291	52·7	284
Experiments on 5½-inch Cast-Iron Revealer as above, but with Inner Wall and						
7	34	Saturated steam, internal surface varnished . . cold walls condensing	48·5	279	26·0	242

[Continued on page 294.]

continued.

22	23	24	25	26	27	28	29	30	31
in Revealer, Diagrams.		Temperature in Steam Pipe Minus Final Temperature = Range of Temperature in Revealer. ° Fahr.	Actual Temperature (taken by Thermometer).			Total Volume of Steam in Revealer and Total Surface (internal) exposed to same, per Stroke.			Steam Condensed in Revealer up to Release in lbs. per Hour per Square Foot of Internal Surface. Lbs.
Final Pressure at end of Exhaust.			Temperature of Wall in 1½-Millimetre hole.		Average Temperature of whole Wall. ° Fahr.				
Lbs. Absolute.	Degrees Fahrenheit.		Average Temperature. ° Fahr.	Hotter or Colder than Initial temperature. ° Fahr.		Volume in Cubic Feet.	Surface in Square Feet.	Ratio $= \frac{\text{Surface}}{\text{Volume}}$.	
long. All Metal Walls. See Figs. 13, 14, 15 *and* 16, *Plate* 7, *for Mean Indicator Diagrams and* 25, 26, 27 *and* 28, *Plate* 8.									
4·6	159	135·8	218	76 colder	217				43·3
15·0	213	81·2	238	57 colder	236				33·5
2·2	130	167·1	327	30 hotter	329				13·8
15·0	213	83·8	330	33 hotter	325	0·009219	0·62	67·2 to 1	17·4
4·9	162	70·3	178	53 colder	176				14·7
15·1	213	41·0	227	27 colder	225				10·0
6·9	176	119·3	233	63 colder	226				25·2
15·3	214	81·6	252	43 colder	245				20·0
Walls. See Figs. 17, 18, 19 *and* 20, *Plate* 7, *for Mean Indicator Diagrams and* 34, 35 *and* 36, *Plate* 8.									
9·9	193	101·3	230	64 colder	229				16·5
16·0	216	78·5	244	51 colder	242				14·6
5·2	164	132·0	329	32 hotter	327				5·10
15·2	214	82·4	334	38 hotter	316				5·00
2·5	135	160·4	281	assuming temp. of saturated steam 14 colder	278	0·04411	1·28	29 to 1	3·78
14·5	211	84·3	266	30 colder	265				7·54
3·3	146	149·8	354	59 hotter	357				2·96
14·9	213	83·8	345	49 hotter	348				1·36
Top and Bottom Covers Varnished.									
8·4	185	110·7	227	52 colder	227	0·04477	varnished 0·84 unvar. 0·49 total 1·33	29·7 to 1	13·3

[*Continued on page* 295.]

TABLE I.—

1 (No. of Revealer, or Cylinder.)	1	2			18	19	20	21
					Pressures and Corresponding Temperatures as taken from Indicator			
					Initial pressure.		Release Pressure.	
No. of Revealer, or Cylinder.	Experiment No.	Conditions of Experiments.			Lbs. Absolute.	Degrees Fahrenheit.	Lbs. Absolute.	Degrees Fahrenheit.
Experiments on same 5½-inch Cast-Iron Revealer, but with 24 and 12 Radial Plates, both Varnished and								
8	35	Saturated steam	Experiment with 24 radial plates not varnished	cold walls condensing	19·4	226	17·0	219
	36		Experiment with 12 radial plates not varnished	cold walls condensing	25·5	241	19·6	227
	37		Experiment with 12 radial plates varnished . .	cold walls condensing	34·3	258	22·0	233
Experiments on same 5½-inch Cast-Iron Revealer, but with 1 or 2 Inner Rings of								
9	38	Saturated steam	With large inner ring . . .	cold walls condensing	29·9	250	19·9	227
	39		With small inner ring . . .	cold walls condensing	34·5	258	19·9	227
	40		With both rings	cold walls condensing	29·0	248	17·9	222

continued.

22	23	24	25	26	27	28	29	30	31
in Revealer, Diagrams.		Temperature in Steam Pipe Minus Final Temperature = Range of Temperature in Revealer. ° Fahr.	Actual Temperature (taken by Thermometer).			Total Volume of Steam in Revealer and Total Surface (internal) exposed to same per Stroke.			Steam Condensed in Revealer up to Release in lbs. per Hour per Square Foot of Internal Surface.
Final Pressure at end of Exhaust.			Temperature of Wall in 1¼-Millimetre hole.		Average Temperature of whole Wall. ° Fahr.				
Lbs. Absolute.	Degrees Fahrenheit.		Average Temperature. ° Fahr.	Hotter or Colder than Initial temperature. ° Fahr.		Volume in Cubic Feet.	Surface in Square Feet.	Ratio $= \frac{\text{Surface}}{\text{Volume}}$.	
$\frac{1}{16}$ inch thick inside Revealer. With Internal Surface of Revealer, Covers and Plates not Varnished.									
9·8	192	103·9	209	87 colder	208	0·03548	taking both sides of radial plates 3·42	96·4 to 1	7·4
9·75	192	104·1	218	78 colder	217	0·03933	2·37	60·2 to 1	10·2
8·0	183	113	223	73 colder	222				9·6
Cast-Iron ⅝ inch thick inside Revealer. Internal Surface not Varnished.									
7·2	178	118	215	81 colder	214	0·03157	1·77	56 to 1	13·2
7·8	182	114·6	217	79 colder	215	0·03577	1·50	42 to 1	15·2
9·7	192	104·7	214	82 colder	214	0·02281	2·16	95 to 1	11·7

TABLE II.—22 EXPERIMENTS ARRANGED ACCORDING TO THE AMOUNT OF CONDENSATION OF STEAM UP TO RELEASE IN LBS. PER HOUR PER SQUARE FOOT OF INTERNAL SURFACE.

Lbs. Water Condensed per Hour per Sq. Ft. of Internal Surface.	Experiment No.	Cylinder Walls.	Diameter of cylinder.	Non-Condensing or Condensing.	Hot or Cold Walls.	Saturated or Superheated steam.	Temperature of Wall in 1½ Millimetre Hole.	Remarks.
lbs.			Ins.				°Fah.	
1·36	22	Cast-iron.	5½	Non-con.	Hot.	Sup.	348	Hottest walls. Superheated steam. Largest cylinders.
2·91	32	Ph. Bronze.	5½	Con.	Hot.	Sup.	341	
2·96	21	Cast-iron.	5½	Con.	Hot.	Sup.	354	
3·78	19	Cast-iron.	5½	Con.	Cold	Sup.	281	
5·00	18	Cast-iron.	5½	Non-con.	Hot.	Sat.	334	
5·10	17	Cast-iron.	5½	Con.	Hot.	Sat.	329	Hotter walls. Saturated steam.
7·25	30	Ph. Bronze.	5½	Non-con.	Hot.	Sat.	335	
7·54	20	Cast-iron.	5½	Non-con.	Cold.	Sup.	266	
9·12	29	Ph. Bronze.	5½	Con.	Hot.	Sat.	334	
9·91	31	Ph. Bronze.	5½	Con.	Cold.	Sup.	273	
13·8	5 & 6	Cast-iron.	2	Con.	Hot.	Sat.	327	Cooler walls. Saturated steam.
14·6	16	Cast-iron.	5½	Non-con.	Cold.	Sat.	244	
16·5	15	Cast-iron.	5½	Con.	Cold.	Sat.	230	
17·4	7 & 8	Cast-iron.	2	Non-con.	Hot.	Sat.	330	
21·3	28	Ph. Bronze.	5½	Non-con.	Cold.	Sat.	238	
22·2	33	Double glass, steam-jacketed	2¾	Con.	Hot.	Sat.	..	
24·2	27	Ph. Bronze.	5½	Con.	Cold.	Sat.	227	
24·2	14	Single Glass.	2	Non-con.	Cold.	Sat.	..	Coolest walls, and smallest cylinders.
26·6	23	Double glass, air-jacketed.	2¾	Con.	Cold.	Sat.	..	
33·5	3 & 4	Cast-iron.	2	Non-con.	Cold.	Sat.	238	
33·7	13	Single Glass.	2	Con	Cold.	Sat.	..	
43·3	2	Cast-iron.	2	Con.	Cold.	Sat.	218	

(*Paper No. 2739.*)

"Experiments on the Filtration of Sewage."

By SIDNEY RICHARD LOWCOCK, Assoc. M. Inst. C.E.

BEFORE describing these experiments it may be useful to consider briefly the views until recently held on filtration, and the results of previous investigations which bear on the subject, in order that the object of these trials and the rationale of the process adopted may be appreciated.

The purification of water by filtration, the purification of sewage by irrigation or filtration, and what is known as the self-purification of flowing water, are processes so closely related, that investigations into the causes and action of any one of them apply more or less to all;[1] and it is therefore unnecessary to consider separately the views that have been held on these subjects individually.

The filtration of water through sand for dietetic purposes, has been long practised, and until about 1859 was generally considered to involve a mere mechanical straining, by which matters suspended in the water were separated and retained in the filter. The purification of sewage by its application to land was believed to be effected in a similar way, the action being supposed to be assisted by the roots of the plants arresting and assimilating the organic matters.[2] Nevertheless the action of air on the organic matter contained in flowing water, and the existence of two kinds of decomposition, putrefactive and non-putrefactive, had long been recognised;[3] and it was known that the salts present in the soil were, to some extent, at any rate the result of the decomposition of organic matter, and that these salts formed the necessary food for plant life.

It has been proved by the Rivers Pollution Commissioners, and by other investigators, both before and after them, that the chemical treatment of sewage can only remove suspended matters,

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[1] Journal of the Roy. Agricultural Soc., 1891, p. 702.

[2] Minutes of Proceedings Inst. C.E., vol. xxvii. pp. 34, 47.

[3] Report to the Metropolitan Board of Works upon the Main Drainage of the Metropolis, 1858, p. 66.

and does not remove the highly putrescible soluble constituents from the liquid which passes off, containing about six-sevenths of the total manurial value of the sewage. The only practicable way of removing these soluble matters is by the application of the liquid to thoroughly aërated porous soil. The late Mr. T. Hawksley,[1] Dr. Letheby,[2] Dr. Frankland,[3] Dr. Voelcker[4] and others have shown that filtration and irrigation have a chemical as well as a mechanical action, and that this action can be maintained if the oxidising powers are not overtaxed, and sufficient intervals of time for thorough aëration are permitted to elapse between the successive applications of the sewage, and if the pores of the land are not allowed to become choked.[5] Unfortunately, however, this aëration presents great difficulty in the treatment of sewage on land.

The researches of Messrs. Schloesing and Müntz,[6] J. Soyka, R. Warington,[7] Downes and Blunt, Dr. Monro,[8] Dr. Tidy,[9] Dr. P. Frankland[10] and others have shown that the conversion of the carbonaceous and nitrogenous organic matters in sewage into harmless inorganic salts is due to the action of micro-organisms, and that this action takes place most rapidly in thoroughly aërated porous soil; that the presence of a base is necessary; that, before nitrification can be set up, the nitrogenous organic matter must be decomposed and converted into ammonia; that complete nitrification can only take place in the dark; that the presence of undecomposed organic matter hinders this process, and that the period of actual nitrification increases with the concentration of the ammoniacal solution operated upon, and, to a certain point, is greatly diminished by rise of temperature.

Messrs. Winogradsky, Warington and Frankland have succeeded in isolating the organisms that produce nitrification, while Mr. T. Leone[11] found that de-nitrification is also the work of bacteria, and takes place in the absence of a sufficient supply of air and in

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[1] Minutes of Proceedings Inst. C.E., vol. xxvii. p. 16.

[2] *Ibid*, p. 37. [3] *Ibid*, vol. xxxii. p. 377.

[4] Dr. Voelcker, p. 401.

[5] Minutes of Proceedings Inst. C.E., vol. xlix. p. 191.

[6] Journal of the Chemical Society, vol. xxxiv. p. 597; Comptes rendus, vol. lxxxix. pp. 891, 1074.

[7] Journal of the Society of Arts, vol. xxx. p. 532; Journal of the Chemical Society, vol. xxxiii. p. 44; vol. xlv. p. 637.

[8] Journal of the Society of Chemical Industry, vol. iv. p. 12.

[9] Journal of the Society of Arts, vol. xxxiv. p. 1127.

[10] "Our Secret Friends and Foes," 1893, p. 89.

[11] Journal of the Chemical Society, vol. lxix., Part II, p. 101.

the presence of organic matter. Following these discoveries, Mr. Dibdin[1] has shown that antiseptic processes of sewage treatment, which it is pretended produce a sterile effluent, are wrong in principle.

The experiments of the Massachusetts Board of Health have carried the matter a step farther, and have proved[2] that, in the intermittent filtration of sewage, the best results are obtained in mature filters; that a distinct regimen is essential to success; that free oxygen is indispensable; that sewage is best purified when held in thin films upon or between grains of sand and gravel; and that the period of greatest destruction of the ordinary sewage bacteria corresponds closely with the time of most active nitrification. In the twenty-third annual report (1891) of the same board (p. 428) the results of further experiments show that:—

> "The purification of sewage by intermittent filtration depends upon oxygen and time; all other conditions are secondary. Temperature has only a minor influence; the organisms necessary for purification are sure to establish themselves in a filter before it has been long in use. Imperfect purification for any considerable period can invariably be traced either to a lack of oxygen in the pores of the filter, or to the sewage passing so quickly that there is not sufficient time for the oxidation processes to take place. Any treatment which keeps all particles of sewage distributed over the surface of sand particles, in contact with an excess of air for a sufficient time, is sure to give a well oxidized effluent, and the power of any material to purify sewage depends almost entirely upon its ability to hold the sewage in contact with air. It must hold both sewage and air in sufficient amounts."

The Author therefore considers the following conclusions have been established:—

1. That filtration is not only a mechanical but also a chemical and biological process, when it is properly carried out and when sufficient aëration is provided for. 2. That no practicable chemical process yet devised will, alone, do more than remove the suspended matters in sewage, and a very small portion of the dissolved impurities. 3. That dissolved impurities can only be removed by the action of micro-organisms, *i.e.*, by nitrification; and this can only be effected subsequently to the decomposition of the organic matter and the formation of ammonia, and in the absence of undecomposed organic matter. 4. That organic matter cannot be destroyed or converted into plant food until it has been dissolved. 5. That, as nitrification proceeds far more rapidly in a moistened and aërated porous soil than in a liquid, and as the

[1] Minutes of Proceedings Inst. C.E., vol. lxxxviii. p. 161.
[2] Report of the State Board of Health, Massachusetts, 1890.

nitrifying powers of soil are capable of cultivation, the process should be carried on by means of filtration, so that the organisms can be cultivated and supplied with food in the filter. 6. That the most important factor in the processes of decomposition and nitrification, and the subsequent preservation of the nitrates formed, is an ample supply of air. 7. That the suspended matters in sewage should not be allowed to pass on to the land or filter-bed, as they clog the surface and have to remain until they are decomposed before they can be destroyed.

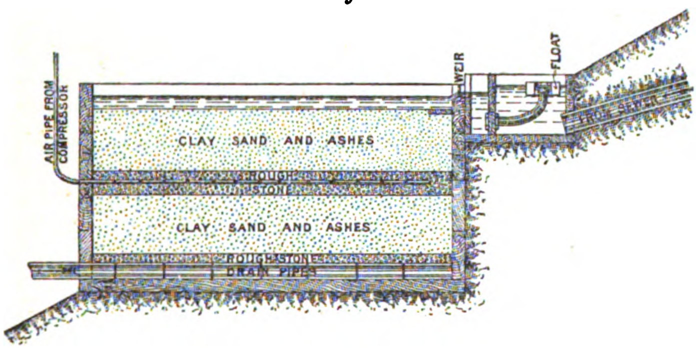
As it is the duty of the engineer to imitate, as far as possible, the processes of nature and to utilize natural forces, it will be well to consider first the cycle of operations which takes place when sewage or other manure is applied to natural soil under ordinary conditions. This action is as follows:—The organic matter is decomposed by the action of micro-organisms, and is dissolved by the rain by which it is carried in solution into the soil, where it is converted by bacteria into nitrites and nitrates, and is then assimilated by vegetation. This decomposition and the formation of nitrites takes place at or near the surface in the presence of light and air, and the subsequent conversion of nitrites to nitrates takes place at a short distance below the surface, and in the dark. The probable reason for the absence of nitrifying organisms at greater depth is the absence of air or of material suitable for nitrification. The soil will not retain the nitrates formed, any excess beyond the requirements of vegetation being washed out, and passing away with the effluent water.

It is therefore evident that, whether the organic matter in sewage be first deposited in the form of sludge and subsequently applied to the land, or be dissolved and passed on to the land as effluent from the tanks, the same process has ultimately to be carried out; and it must be a question of convenience and cost as to whether it is better to precipitate the organic matter in suspension, and transport it to some place where it can be made use of or thrown away, or to endeavour to dissolve the whole of the organic matter in suspension and treat it in the effluent water so as to produce nitrates. Such an effluent, containing the whole of the manurial value of the sewage, could then be used either for irrigation when required, or turned into a stream without nuisance when not wanted for the land; as it is well known that its application is only useful at certain stages of the growth of vegetation, and its enforced use at other times is one of the most fruitful sources of failure in sewage farming. The remaining precipitate from the tanks would then only consist of

mineral matters, and such small portion of organic matters as are practically insoluble, and could readily be got rid of or subsequently used as a filtering medium. In either case, it must be a question for chemists to decide what chemicals are most suitable in each individual case. The Author would suggest that lime, possibly in combination with some other chemical or chemicals, is a most suitable agent; as it acts first as a precipitant and solvent of organic structures, and combines with the carbonic acid produced by their decomposition in the presence of air, forming the base necessary for the process of nitrification.

The experiments made by the Author were undertaken to ascertain the possibility of constructing and working a filter that should follow the operations of nature, and promote the growth

Fig. 1.



Scale ¼ inch = 1 foot.

of the nitrifying organisms and the consequent purification of a sewage effluent, when working continuously.

In the first experiment the filter was constructed as shown in *Fig. 1.* The tank was made of wood, tongued-and-grooved so as to be perfectly water-tight, 7 feet 6 inches square by 4 feet deep, measured inside. The filter was formed by laying on the bottom of the tank, along its centre-line, a row of 3-inch agricultural drain-pipes with open joints, and covering them and the whole of the bottom of the tank with a 6-inch layer of $\frac{5}{8}$-inch broken stone. On this stone was laid 15 inches thick a mixture of the natural soil of the district, which is a fairly stiff clay, with fine engine-ashes and building-sand, in the proportion of 2 parts of soil to 1 of ashes and 1 of sand. The next portion consisted of 6 inches of rough stone similar to the bottom layer. In the middle of this layer, a $\frac{3}{4}$-inch wrought-iron pipe, closed at one end and

pierced along both sides with holes $\frac{1}{16}$-inch in diameter, 6 inches from centre to centre, was laid, the outer end being connected with the discharge-pipe of a small air-compressor. The latter was driven by a belt from a water-wheel which is turned by the sewage and serves to mix and regulate the discharge of the chemicals used in treating it. The air was discharged by the perforated pipe into the layer of rough stone which served to distribute it equally throughout the body of the filter. The upper layer of sand, clay and ashes, 15 inches thick, is similar to the bottom one already described, and the total depth of the filtering material is thus 3 feet 6 inches. The crude sewage, after being roughly screened, was conveyed to the filter by a small branch pipe from the main sewer, which discharged into a wooden box fitted with a floating outlet carrying an indiarubber pipe, so arranged that the float could be weighted to give a regular discharge on to the filter at any desired rate. This floating outlet delivered into a second compartment of the box, from which the sewage flowed on to the filter over a small gauging-weir. The filter was supplied continuously with sewage and air, the sludge deposited in the float-box and inner compartment was cleaned out from time to time, and the surface of the filter was raked over once a week.

The operation of the filter was started on the 4th May, 1892, the sewage passing over the weir at the rate of 263,780 gallons per acre per twenty-four hours. At this rate it worked continuously for nineteen days, the surface becoming gradually covered with deposit from the sewage, which at the end of this time stood about 5 inches deep over the surface, while the rate of flow was reduced to 100,000 gallons per acre per day. The effluent during the whole of the time was clear and colourless, and, after the first few days, odourless. From the 23rd May to the 30th May, the flow of sewage was stopped, as the sewage was diverted to another part of the farm and could not be brought to the filter. On the 31st May the filter was well raked over and re-started at the same rate, *i.e.*, 263,780 gallons per acre per day.

On the 14th June samples of the sewage and effluent were taken, and gave on analysis results (see Table, p. 303), which showed a reduction of 99·1 per cent. in the free ammonia and 98·5 per cent. in the albumenoid ammonia, or 98·93 per cent. in the sum of the ammonias.[1] The filter had been working

[1] All the analyses were made according to Wanklyn's method—those of the samples taken on the 14th and the 19th June by Mr. Alex. E. Tucker, F.C.S., and the remainder by the Author.

altogether thirty-three days, and the effluent was perfectly clear, colourless, odourless and tasteless, and had been used for drinking by the men working on the farm. The average air-pressure was 0·6 inch of water, and the volume 2·99 cubic feet of free air per cubic foot of sewage, calculated on the rate of flow at which the filter was started. By the 28th June, the surface of the filter had

—		Sewage.	Effluent.
Free ammonia. . . .	parts per 100,000	4·65	0·04
Albumenoid ammonia .	" "	2·40	0·036
Sum of ammonias .	" "	7·05	0·076

become much choked, and the flow of the sewage on to it was reduced to 66,900 gallons per acre per day; but the effluent preserved its satisfactory appearance and the absence of smell and taste. On the 29th June, the supply of sewage was stopped, and, as soon as the surface was sufficiently dry, the top was removed to a depth of 2 inches and was replaced with fresh clay and sand. The sewage was supplied again on the 1st July at the rate of 263,780 gallons per acre per day. On the 7th July, after the filter had been at work for fifty-four days, another sample of the effluent was taken, the analysis of which gave

	Parts per 100,000.
Free ammonia	0·013
Albumenoid ammonia	0·024
Sum of ammonias	0·037

On the 10th July, the surface had become slightly clogged and the quantity flowing on to it had to be reduced. On the 12th July, the effluent was apparently as good as on the 7th, and on this date the supply of air to the filter was stopped. The effect of this on the effluent was immediately apparent, as on the 14th it had become brownish and opalescent, though it had no smell. On the 19th July, the effluent was pale brown in colour, with a slight smell, and was full of light fungoid growth. By this time the surface of the filter was almost entirely choked, and the quantity of effluent was very small, though considerable nitrification was still being effected, as an analysis gave—

	Parts per 100,000.
Free ammonia	0·212
Albumenoid ammonia	0·066
Sum of ammonias	0·278

On the 20th July, the tank was removed and refixed below the precipitating-tanks, to try the effect of the filtration on the tank-effluent after treatment with 6 grains of lime and 12 grains of Spence's alum per gallon, which was the treatment adopted on the works at that time. In removing the filtering material, after the clogged surface had been scraped off, a careful examination showed that, to all appearance, the material was quite clean and unaffected by the passage of the sewage through it.

After the tank had been fixed in its new position, the old material was replaced in the same manner as before, and the filter was worked in exactly the same way, except that the float and pipe were removed and the tank-effluent was brought from the outlet-channel from the tanks to the filter by a siphon, the quantity supplied being regulated by a cock. The filter was again started on the 12th August at the rate of 263,780 gallons per acre per day, the pressure and volume of air being the same as before. A sample of the liquid running on to the filter was taken on the 23rd of August; but, owing to accidents, the sample of tank-effluent of the 23rd and that of the filter-effluent of the 31st do not strictly compare. As, however, the precipitating-tanks were only changed on the 1st of each month, the tank-effluent on the 30th must have been far worse than that on the 23rd August. The analyses gave:—

—		Tank Effluent.	Filter Effluent.
Free ammonia. . . .	parts per 100,000	4·530	0·0112
Albumenoid ammonia .	,, ,,	0·052	0·0325
Sum of ammonias .	,, ,,	4·582	0·0437

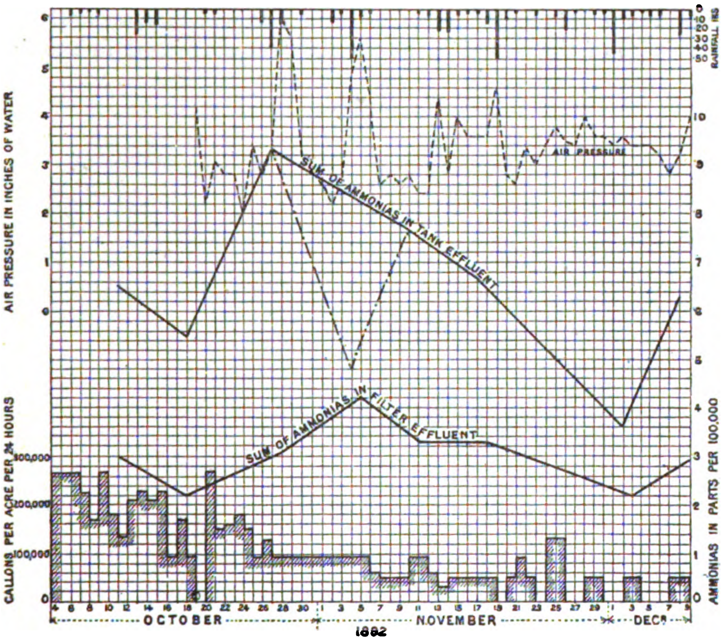
The rate of flow was about 186,000 gallons per acre per day. The filter had then been at work continuously for nineteen days, and the same filtering material had been used for eighty-eight days. The effluent was clear, bright, colourless and odourless.

On the 1st September the sewage was diverted from the tanks to irrigate the upper part of the farm, and consequently the filter had again to be stopped, and the sewage continued to be used for irrigation throughout the month. At this time also certain difficulties occurred which led to the air-compressor being dispensed with at the end of the month. The Author then determined to try

the effect of a blower in place of the compressor, and, during the time which elapsed while the blower was being procured and fixed, it was decided to allow the filter to continue at work without any air being forced in. The tank-effluent was therefore supplied on the 4th of October at the rate of 263,780 gallons per day per acre as before, and a record of the observations made is given in the Appendix, Table I, and in the form of a diagram, *Fig. 2.*

From these it will be seen that the filter quickly became clogged, until on the 16th and 18th October, the tank-effluent could only

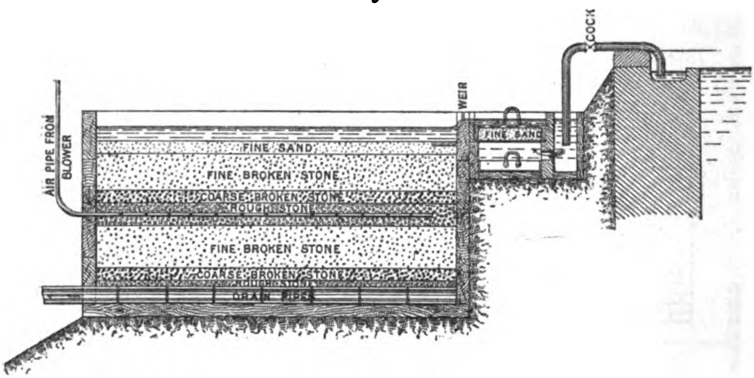
Fig. 2.



be applied at the rate of 93,000 gallons per acre per day. On the 19th October the surface of the filter, which had been allowed to become fairly dry, was well raked over, and the new blower was started to work. The rate of flow was temporarily increased, but the quality of the effluent showed no improvement. The tank-effluent, up to the 15th November, had been at all times practically neutral; but from this date more lime was added to the sewage, making the total quantity 11·9 grains per gallon, and rendering the tank-effluent slightly alkaline. This did not, however, produce any beneficial effect on the filter, as, until the experiment was

concluded on the 9th December, the ammonias of the filter-effluent practically varied with those in the tank-effluent, especially if the sample taken on the 4th November is disregarded, owing to its dilution having been due to a heavy storm of rain. The albumenoid ammonia was reduced in a far higher degree than the free ammonia, showing that the reduction was probably due to the retention and destruction of the matters appearing in this form in the body of the filter—the reverse of what takes place when the air supplied to the filter is promoting its action most efficiently. On the 9th December the filter having become practically completely choked, the filtering material was taken out; when it was found that it was full of decomposing matter, and that, owing to the subsidence of the material, the air-pipe was embedded in the fine

Fig. 3.



Scale ¼ inch = 1 foot.

material above the layer of rough stone. The lower half of the pipe was full of sewage matter, and in all probability no air had passed into the body of the filter during this experiment.

At the beginning of February, the tank was refilled with coarser materials arranged as shown in *Fig. 3*, in order to observe the result when passing larger quantities of the tank-effluent. The bottom layer was formed of 6 inches of rough stone, broken to pass a sieve with 1-inch meshes, laid over the drain-pipes and the floor of the tank. Upon this was placed a 3-inch layer of finer broken stone, broken to pass ⅝-inch meshes, and this supported 10½ inches of screenings from the broken stone, passed through a sieve having 70 meshes to a square inch. The air-pipe was laid in a 6-inch layer of the rough stone as before, and upon this was laid 3 inches of ⅝-inch broken stone, and 9 inches of the 70-mesh

screenings. The top layer was formed of 3 inches of building-sand, the object being to regulate the flow in such a way that the top layer of sand would become saturated, and only allow the liquid to pass from it over the coarser material below in thin films. The total depth of the filtering material was thus 3 feet 1½ inch. The broken stone was obtained from a quarry at North Malvern, and the sand from Kidderminster. The float-box was removed, and in its place was fixed a small tank, fitted with a partition. The siphon discharged into the smaller compartment thus formed, and from it the liquid to be filtered passed into the larger compartment and upwards through a filter of 3 inches of building-sand (1 square yard in area), supported on fine copper gauze fixed to the bottom of a movable tray. Beneath this tray, a movable pan was fitted to catch any sludge that might be deposited. After passing the upward filter, the liquid flowed over the small weir on to the experimental filter as in the former operations.

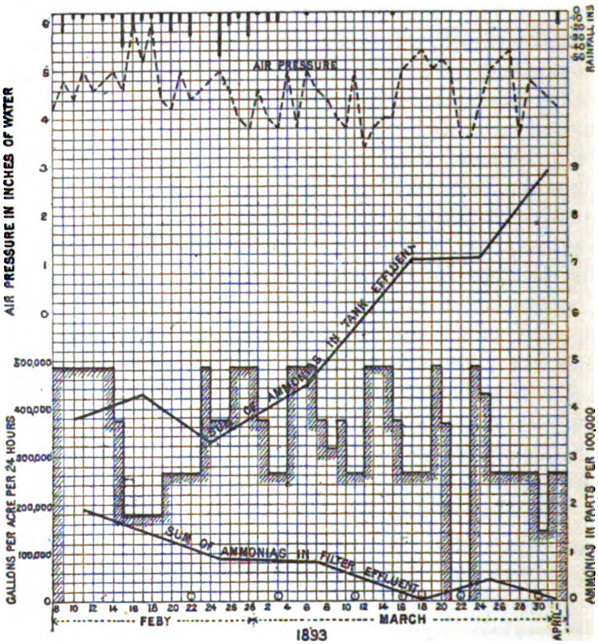
This arrangement would, it was hoped, arrest the matters which choked the surface, so that they could be removed from time to time without interfering with the main filter; but nothing was ever retained on the under-side of the upward filter or deposited in the sludge pan. The upper surface of this filter became coated over at the same time, and apparently with the same material, as the surface of the main filter.

On the 8th February, 1893, the filter was again started with the effluent from the tanks applied at the rate of 484,000 gallons per acre per day; and the observations made are shown in Table II, Appendix, and as a diagram *Fig. 4*. For four days the liquid disappeared as soon as it reached the surface of the filter, but on the 12th February it began to spread evenly over the surface, and on the 14th the quantity applied was reduced to 373,890 gallons per acre per day, and on the 15th to 178,390 gallons. The surface of the filter then showed a film of deposit, but, after working for four days at this rate, it cleared itself to some extent, and the rate of flow was increased to 263,780 gallons per acre for the next four days. On the 22nd February the surface was raked over, and the flow increased. The surface was also raked over on 3rd, 11th and 18th March. On the 21st March, after the filter had been at work forty-one days, the surface appeared to be so dirty that the supply of tank-effluent was stopped, and one inch of the surface-sand was skimmed off and replaced with fresh sand. On the 30th March the surface was again raked over. After April 1st, the experiments had to be stopped owing to the illness of the assistant who had charge of them; but they had been

carried on for a sufficient time to enable some definite conclusions to be drawn from them.

After the experiments were concluded, the Author made a careful examination of the filtering-material, and found that, except at the actual surface, it appeared to have undergone no change whatever, and only emitted an ordinary earthy smell, being perfectly clean. The diagrams, *Fig. 4*, show the quantities of tank-effluent passing on to the filter, expressed in gallons per acre per twenty-four hours; the combined ammonias of the tank-

Fig. 4.



Dates when surface raked over marked O .

effluent and of the effluent passing from the filter; the air-pressure expressed in inches of water; and the rainfall, for the measurement of which the Author is indebted to the Rev. G. S. Munn, of Madresfield Rectory.

Fig. 4 shows that, notwithstanding considerable variation in the quantities of liquid applied, while the ammonias in it rose from 3·80 parts per 100,000 on the 10th February to 8·92 parts on the 31st March, the ammonias in the effluent show a steady decrease t) the 18th March, when they amounted to only 0·03 part per

100,000, or 0·42 per cent. of the ammonias in the liquid applied. The application of an increased quantity on the 23rd and 24th March produced a temporary disturbance, and on the 25th the ammonias in the effluent rose to 0·44 part per 100,000 but on the 1st April they had again fallen to 0·054 part, *i.e.*, to 0·67 per cent. of those in the liquid applied to the filter.

None of these effluents when kept in a warm room have undergone putrefactive decomposition, but after some time have developed a green growth on the bottom of the stoppered bottles in which they have been kept. The experiments confirm the conclusions arrived at in the Massachusetts experiments, that the filter does not reach its condition of greatest efficiency for some little time after it is started, and that the efficiency is impaired by variations in the quantity of liquid applied. The average quantity applied during the period covered by Table II (Appendix) is equal to 353,800 gallons per acre per day, the average air-pressure being 4·5 inches of water. The quantity applied when the most satisfactory results were obtained was at the rate of 263,780 gallons per acre per day; so that at this rate the area required per million gallons of effluent of the same impurity as that experimented with, would be 3·8 acres with a depth of 3 feet. As, however, under this system the purification is independent of the action of the surface, it would appear probable that, by increasing the depth, the quantity purified would also be increased, and so a much smaller area would be sufficient. The dry-weather flow of the sewage experimented upon is 16 gallons per day per head of the population, so that the quantity treated at the most efficient rate is equal to that from 16,486 persons per acre.

The practical difficulty with all filtration is the clogging of the surface, but this affects the quantity passed in an inverse ratio and the degree of purification directly; that is, the greater the clogging of the surface, the more slowly the liquid passes through the filter and the greater the consequent purification. It would appear, however, from a comparison of the experiments, that this clogging does not take place so quickly when air is being supplied to the body of the filter as when it is not; and this is probably due to the matters which produce the clogging being broken up and destroyed in the presence of the air directly they pass below the surface. Frost would not affect the action of the filter, as the surface is always covered with liquid, except in so far as it would to some extent retard the process of nitrification, and this could be obviated by warming the air supplied. Nitrifi-

cation would probably be accelerated by this means at all times, but the Author has up to the present had no opportunity of experimenting on this point.

The distribution of the sewage over the surface is of course uniform, and each part of the filter has its proper share of work. The aëration is under control, and the pressure of air required is very small, but it must not be allowed to rise so as to form blow-holes by which the efficiency of the filter may be impaired or destroyed. It may be noticed that the average of the combined ammonias in the liquid used (Table II, Appendix) is 2½ times as great as the average of the combined ammonias in the sewage used in the Massachusetts experiments, and is slightly less than the average found in the sewage from water-closeted towns, as given by the Rivers Pollution Commissioners. These experiments have been made with a specially-prepared filter, but there is every reason to suppose that where suitable ground is available, by laying perforated air-pipes in the open sub-soil and aërating it as described, the efficiency of such soil for the purification of sewage may be much increased. Where such soil is not available, the most rational process would appear to be to treat the tank-effluent in a specially prepared filter, to use the effluent from the filter for irrigation when required, and at other times to turn it into the stream.

In conclusion, the Author desires to acknowledge his indebtedness to Colonel W. Stallard for affording the necessary facilities for the experiments to be carried out.

The Paper is accompanied by tracings from which the *Figs.* in the text have been prepared.

APPENDIX.

TABLE I.

Date.	Rate of Flow on to Filter in Gallons per Acre per 24 Hours.	Air-Pressure in Inches of Water.	Rainfall, Gallons per Acre per 24 Hours.	Tank Effluent.		Filter Effluent.	
				Free Ammonia.	Albumenoid Ammonia.	Free Ammonia.	Albumenoid Ammonia.
1893				Parts per 100,000.		Parts per 100,000.	
Oct. 4	263,780	..	..				
" 5	263,780	..	..				
" 6	263,780	..	1,584				
" 7	221,085	..	679				
" 8	164,158	..	452				
" 9	263,780	..	905				
" 10	178,390	..	..				
" 11[1]	133,611	..	..	6·40	0·084	2·93	0·08
" 12	206,703	..	..				
" 13	221,085	..	6,108				
" 14	206,853	..	3,393				
" 15	221,085	..	4,072				
" 16	93,000	..	..				
" 17	164,158	..	..				
" 18[2]	93,000	..	..	5·20	0·32	2·133	0·08
" 19	..	4·5	..				
" 20[3]	263,780	2·1	226				
" 21	149,926	3·1	452				
" 22	157,042	2·8	..				
" 23	178,390	2·8	..				
" 24	149,926	2·0	..				
" 25	93,000	3·3	..				
" 26	127,156	2·8	3,167				
" 27[4]	93,000	3·5	8,596	9·00	0·40		
" 28[5]	93,000	6·0	3,167	..	..	3·00	0·106
" 29	93,000	5·6	..				
" 30	93,000	3·1	452				
" 31	93,000	2·8	..				
Nov. 1[6]	93,000	2·6	..				
" 2	93,000	2·1	2,941				
" 3	93,000	2·6	226				

[1] Tank-effluent.—Fœtid, brown, full of flocculent matter, some precipitate. Filter-effluent.—Fœtid, pale brown, full of flocculent matter.

[2] Tank-effluent.—Fœtid, brown, full of flocculent matter, some precipitate. Tanks changed. Filter-effluent.—Cloudy, fœtid, with some fungoid growth. Surface of filter coated over with deposit.

[3] Surface of filter raked over on 19th.

[4] Fœtid, cloudy, brown, with some precipitate.

[5] Slightly fœtid, cloudy, slight precipitate.

[6] Tanks changed.

TABLE I—*continued.*

Date.	Rate of Flow on to Filter in Gallons per Acre per 24 Hours.	Air-Pressure in Inches of Water.	Rainfall, Gallons per Acre per 24 Hours.	Tank Effluent.		Filter Effluent.	
				Free Ammonia.	Albu-menoid Ammonia.	Free Ammonia.	Albu-menoid Ammonia.
1892				Parts per 100,000.		Parts per 100,000.	
Nov. 4[1]	93,000	4·8	11,763	4·00	0·80		
" 5[2]	93,000	5·6	2,036	..	..	4·00	0·23
" 6	62,000	4·3	905				
" 7	46,500	2·6	..				
" 8	46,500	2·8	..				
" 9	46,500	2·6	..				
" 10[3]	93,000	2·8	..	7·00	0·68		
" 11[4]	93,000	2·4	226	..	..	3·12	0·12
" 12	62,000	2·4	1,357				
" 13	31,000	4·2	4,750				
" 14	46,500	2·8	5,203				
" 15[5]	46,500	4·0	1,131				
" 16	46,500	3·6	452				
" 17[6]	46,500	3·6	226	6·50	0·16		
" 18[7]	46,500	3·6	3,393	..	..	3·20	0·05
" 19	..	4·6	11,537				
" 20	46,500	2·8	..				
" 21	93,000	2·6	226				
" 22	46,500	3·4	..				
" 23	..	3·0	452				
" 24	131,890	3·4	..				
" 25	131,890	3·8	1,584				
" 26	..	3·5	4,524				
" 27	..	3·4	226				
" 28	46,500	4·0	..				
" 29	46,500	3·6	452				
" 30	..	3·6	452				
Dec. 1[8]	..	3·4	10,406				
" 2[9]	46,500	3·6	2,262	3·20	0·40		
" 3[10]	46,500	3·4	1,131	..	..	2·12	0·06
" 4	..	3·4	226				
" 5	..	3·4	1,131				
" 6	..	3·2	679				
" 7	46,500	2·8	..				
" 8[11]	46,500	3·2	5,882	6·00	0·24		
" 9[12]	..	4·0	1,131	..	..	2·80	0·06
Average	..	3·4	..	..	..	..	..

[1] Very fœtid, black. [2] Cloudy, slightly fœtid.
[3] Brown, fœtid, full of flocculent matter.
[4] Cloudy, small quantity of flocculent matter. Very slight smell.
[5] Lime added to sewage increased.
[6] Light brown, fœtid, full of flocculent matter.
[7] Clear, small quantity of light brown flocculent matter—slight odour.
[8] Tanks changed. [9] Pale brown, cloudy, full of flocculent matter, fœtid.
[10] Clear, colourless, slight odour, flocculent matter.
[11] Pale brown, cloudy, fine flocculent matter, slightly fœtid.
[12] Cloudy, slight whitish sediment, colourless.

TABLE II.

Date.	Rate of Flow on to Filter in Gallons per Acre per 24 Hours.	Air-Pressure in Inches of Water.	Rainfall. Gallons per Acre per 24 Hours.	Tank Effluent.		Filter Effluent.	
				Free Ammonia.	Albumenoid Ammonia.	Free Ammonia.	Albumenoid Ammonia.
1893.				Parts per 100,000.		Parts per 100,000.	
Feb. 8[1]	484,000	4·2					
„ 9	484,000	4·8	4,524				
„ 10[2]	484,000	4·4	905	3·20	0·60		
„ 11[3]	484,000	5·0	1,131			1·80	0·12
„ 12	484,000	4·6					
„ 13	484,000	4·8	1,357				
„ 14	373,890	5·0	905				
„ 15	178,390	4·6	7,691				
„ 16[4]	178,390	6·0	4,072				
„ 17[5]	178,390	5·2	1,584	4·00	0·32		
„ 18[6]	178,390	6·0	226			1·30	0·12
„ 19	263,780	4·4	2,262				
„ 20	263,780	4·2	4,298				
„ 21	263,780	5·0	2,262				
„ 22[7]	263,780	4·4	5,882				
„ 23	484,000	4·6					
„ 24[8]	373,890	4·8	226	3·00	0·28		
„ 25[9]	373,890	5·0	10,180			0·80	0·06
„ 26	484,000	4·6	2,715				
„ 27	484,000	4·0	1,131				
„ 28	484,000	3·8	5,203				
Mar. 1	373,890	4·6	452				
„ 2[10]	263,780	4·0	2,262				
„ 3[11]	263,780	3·8	2,036				
„ 4	484,000	5·0					
„ 5	484,000	3·8					
„ 6[12]	484,000	5·0	452	4·00	0·48		
„ 7[13]	373,890	4·6	..	..	..	0·80	0·04
„ 8	318,835	4·4					
„ 9	373,890	4·0					
„ 10	263,780	3·8					
„ 11[14]	263,780	5·0					

[1] Filter started. (Tanks changed January 26.)
[2] Pale brown, cloudy, flocculent matter and brown deposit, fœtid.
[3] Effluent clear, odourless, slightly opalescent.
[4] Surface coated over with deposit.
[5] Pale brown, cloudy, flocculent matter and dark deposit, fœtid.
[6] Clear, brilliant, colourless, odourless.
[7] Surface of filter raked over.
[8] Pale brown, cloudy, flocculent matter and brown deposit, fœtid.
[9] Clear, colourless, odourless, brilliant.
[10] Tanks changed.
[11] Surface coated over with deposit. Surface raked over.
[12] Very pale brown, cloudy, flocculent matter and brown precipitate, fœtid.
[13] Clear, colourless, odourless, brilliant.
[14] Surface of filter raked over.

TABLE II—*continued.*

Date.	Rate of Flow on to Filter in Gallons per Acre per 24 Hours.	Air-Pressure in Inches of Water.	Rainfall. Gallons per Acre per 24 Hours.	Tank Effluent.		Filter Effluent.	
				Free Ammonia.	Albu-menoid Ammonia.	Free Ammonia.	Albu-menoid Ammonia.
1893.				Parts per 100,000.		Parts per 100,000.	
Mar. 12	484,000	3·4					
„ 13	484,000	3·8					
„ 14	484,000	4·0					
„ 15	373,890	4·0	679				
„ 16	263,780	5·0					
„ 17[1]	263,780	5·2	..	6·40	0·70		
„ 18[2]	263,780	5·4	..	..	..	0·008	0·022
„ 19	484,000	5·0					
„ 20	363,000	5·2					
„ 21[3]	..	5·0					
„ 22[3]	..	3·6					
„ 23	484,000	3·6					
„ 24[4]	428,945	4·4	..	6·40	0·72		
„ 25[5]	263,780	5·0	..	..	..	0·40	0·04
„ 26	263,780	5·2					
„ 27	263,780	5·4					
„ 28	263,780	3·6					
„ 29	263,780	4·8					
„ 30[6]	142,880	4·6					
„ 31[7]	263,780	4·4	..	8·00	0·92		
Apr. 1[8]	263,780	4·2	2,941	..	..	0·03	0·024
Averages	353,813	4·5					

[1] Very pale brown, cloudy, light flocculent matter, fœtid. Surface coated over with deposit.

[2] Clear, colourless, odourless, brilliant. Surface raked over.

[3] Filter stopped and 1 inch of surface skimmed off and replaced with fresh sand.

[4] Pale brown, light flocculent matter and brown deposit, fœtid.

[5] Clear, very pale yellow tinge, odourless, brilliant.

[6] Surface of filter raked over.

[7] Pale brown, light flocculent matter and deposit, fœtid.

[8] Clear, colourless, odourless, brilliant. Tanks changed.

(*Paper No. 2682.*)

"On Formulas for Pile-Driving."

By Charles Haynes Haswell, M. Inst. C.E.

In the absence of satisfactory experiments or observations whereby the impact of the ram of a pile-driver may be estimated, mathematicians and engineers who have essayed to give formulas for the effect of this operation not only vary materially in their statements, but in some instances have introduced elements but little connected with it. The resistance opposed by a pile to the blow of a ram is the measure of its value to sustain stress, whatever may be its diameter, weight, length, or modulus of elasticity. The diameter and length of a pile do not affect the question; their effect is to limit its penetration. The weight of the pile is worthy of consideration only as affecting the weight of ram employed. The relative elasticity is of little moment; for when a pile approaches the limit of its penetration, its head is dressed off, if broomed, and if split or liable to be so, it is confined by a ring. In fact, the weight of the ram being proportioned to the duty required of it, the diameter, length and elasticity of the pile are inconsiderable, where so great factors of safety, ranging in various formulas, from one-twelfth to one-fifth, are employed.

In the following treatment of the subject, the pile is supposed driven to a final depression not exceeding 0·5 inch per blow, with a level and sound head so far as practicable. Other conditions, such as greater depression than this, and broomed heads, have no positive value; for unless piles are driven to their full capacity, the weight of the load to be sustained will require their number to be proportionately increased, and, in the case of heavy superstructures, this involves setting them at less than a proper distance apart.

In the formulas of Saunders, Rankine, Molesworth, Weisbach, Mason, Trautwine, Nystrom and Wellington, the final depression or penetration, expressed in inches or feet, is given as a divisor. Hence, assuming the impact of a ram to be in a given case 100,000

lbs. and the depression 0·5 inch, and the resistance 200,000 lbs.; if the depression is 0·25 inch, the resistance is 400,000 lbs., and so on to infinity, or positive refusal, and yet the impact has not increased. When a street pavior has depressed a stone 1 inch with his rammer at one blow and 0·5 inch by the succeeding blow, he has not doubled the force with which he has struck it. The effect of one blow is lost to consideration in that which follows it, as the measure of the last impact is the measure of the resistance a pile offers, which is the weight it will bear. The point then is, not how the resistance of a pile is obtained, nor its inertia nor the effect of cushioning; but what the force required to drive it to its final depression, or the measure of its resistance.

Illustration of existing Formulas.

Assume rams of 1,000 lbs. and 2,000 lbs. weight falling respectively 20 and 25 feet; mean section of pile 100 square inches, weight 700 lbs., and length 35 feet; penetration under last blow 0·5 inch, *i.e.* 0·042 foot. Table I gives results computed according to the following formulas, in which

W	is	the weight of the ram in cwt.
w	,,	,, ,, ,, lbs.
F	,,	safe load in cwt.
f	,,	,, ,, lbs.
f'	,,	extreme load in lbs.
D	,,	depression of the pile by a blow in feet.
d	,,	,, ,, ,, ,, inches.
H	,,	fall of the ram in feet.
h	,,	,, ,, ,, inches.
p	,,	weight of the pile in lbs.
l	,,	length ,, ,, feet.
s	,,	sectional area of the pile in square inches.
E	,,	modulus of elasticity of the pile.

1. Saunders. $$\frac{w\,\mathrm{H} \div \mathrm{D}}{8} = f.$$

2. Rankine. $$\sqrt{\left(\frac{4\,\mathrm{E}\,s\,w\,\mathrm{H}}{l} + \frac{4\,\mathrm{E}^2 s^2 \mathrm{D}^2}{l^2}\right)} - \frac{2\,\mathrm{E}\,s\,\mathrm{D}}{l} = w.$$

This formula embraces the modulus of elasticity, length and sectional area of the pile, all of which are held to be unnecessary for the reasons given, especially when the margin of safety is so large

as it is in pile-driving. His rule for extreme load is, "1,000 lbs. per square inch of section, the pile being driven home."

3. Molesworth. $\frac{W\,h'}{8\,d'} = F.$

4. Weisbach. One formula embraces elements like those of Rankine. A second is,

$$\frac{w^2\,H}{D(w+p)} + (w+p) = f'.$$

5. Mason. $\frac{w^2\,h}{D(w+p)} = f'.$

6. Trautwine. $\frac{0 \cdot 023\sqrt[3]{H}\,w}{d+1} \times 2{,}240 = f'.$

7. Nystrom. $\frac{w^3\,h}{6\,d\,(w+p)} = f.$

8. Wellington. $\frac{2\,w\,H}{D+1} = f'.$

TABLE I.—BEARING-POWER OF PILES. DEPRESSION IN ALL CASES, 0·5 INCH.

Formula of	Factors Given by the Authors of the Formulas.	1,000 Lbs. Falling 20 Feet.		2,000 Lbs. Falling 25 Feet.	
		Extreme Load.	Safe Load.	Extreme Load.	Safe Load.
		Lbs.	Lbs.	Lbs.	Lbs.
1. Saunders . . .	8 to 3	476,000	{ 59,500 158,666 }	1,190,000	{ 148,750 396,666
2. Rankine . . .	{ 5 to 1	100,000	{ 20,000 100,000 }	100,000	{ 20,000 100,000
	5 to 1	332,125	{ 66,425 332,125 }	646,246	{ 129,249 646,246
3. Molesworth . .	8 to 0·33	480,080	{ 60,010 160,026 }	1,199,520	{ 149,940 399,840
4. Weisbach . .	100 to 10	281,112	{ 2,811 28,111 }	884,534	{ 8,845 88,453
5. Mason . . .	10 to 4	280,112	{ 28,111 70,028 }	881,834	{ 88,183 220,417
6. Trautwine . .	12 to 2	93,218	{ 7,768 46,609 }	200,592	{ 16,716 100,296
7. Nystrom . . .	6	166,086	27,681	658,434	109,739
8. Wellington . .	..	..	38,387	..	95,970
Average[1].	7·7 to 2·3	..	75,384	..	168,725

The numbers in 4th and 6th columns under safe load show the results computed according to the several formulas with the factors of safety proposed for them.

[1] Omitting Weisbach's factors.

In deciding upon a factor of safety in a formula for pile-driving, the following elements must be considered:—

The friction of the machine; the resistance of the atmosphere to the fall of the ram and the cushioning on the head of the pile, however square it may be dressed off; the want of verticality both in the fall of the ram and in the plane of the pile, and the consequent lateral vibration; the inertia; the vibration and condition of the soil.

From records of some ascertained sustaining-powers of piles,[1] the Author finds that in these cases, the powers range between 2·3 and 3·7 times those which the formulas indicate.

The averages of results deduced by the several formulas for safe loads in the foregoing Table are 4·5 times less than that given by two of the formulas, and 9·7 times greater than given by the least of them, disregarding the extreme one of Weisbach.

New Formula.

In view of this variation, the Author endeavoured to ascertain the difference between the results by the formulas expressed in foot-pounds, and the actual *vis viva* of a falling body; and, although the experiments were on a limited scale, the following results may be of interest:—

Weight.	Fall.	Velocity.	Force of Impact.	Ratio of Impact to Velocity.
Lb.	Feet.	Feet per Sec.	Lbs.	
1	1·0	8·0	32	4·0
1	1·5	9·5	42	4·3
1	2·0	11·31	52	4·6

He further submits the following formula,

$$\frac{4\,w\sqrt{2\,g\,\text{H}},}{\text{C}}$$

or

$$\frac{4\,w\,8\sqrt{\text{H}}}{\text{C}} = l;$$

where w is the weight of ram in lbs., H, the height of fall in feet, g, the value of gravity in kinetic units, and C a constant that varies between the values 3 to 6 according to the nature and con-

[1] "Trautwine's Engineers' Pocket Book," pp. 643–44.

dition of the soil, the character of the piles and excellence of their driving.

Applying the formula to the same elements as those given in the Table, the result would be for a ram of 1,000 lbs. weight falling 20 feet,

$$\frac{4 \times 1{,}000 \times 8 \sqrt{20}}{C} = l,$$

and for a ram of 2,000 lbs. falling 25 feet

$$\frac{4 \times 2{,}000 \times 8 \sqrt{25}}{C} = l.$$

Assuming the value of F to be 3·5, the first expression gives:—

$$\frac{1{,}000 \times 8 \sqrt{20}}{0 \cdot 875} = 40{,}868 \text{ lbs.},$$

and the second gives:—

$$\frac{2{,}000 \times 8 \sqrt{25}}{0 \cdot 875} = 91{,}428 \text{ lbs.}$$

These results are in accordance with the Author's experience. To compare the formula proposed with those already alluded to, assuming a ram of 3,500 lbs. weight falling 9 feet to produce a depression of 0·5 inch by the final blow, taking the value of C as 6, Table II has been calculated:—

TABLE II.

Formulas for Safe Load.	Weight 1,000 Lbs. Fall 20 Feet.	Weight 2,000 Lbs. Fall 25 Feet.	Safe Load.	
			Weight 3,500 Lbs. Fall 9 Feet.	Weight 3,500 Lbs. Fall 9 Feet.
	Units.	Units.	Lbs.	Units.
1. Saunders . . .	5·73	4·5	250,016	4·46
2. Rankine . . .	3·61 11·98	4·13 7·34	100,000 466,562	1·78 8·32
3. Molesworth . .	5·57	4·54	280,000	5·0
4. Weisbach . . .	1·01	1·0	629,200	11·23
5. Mason	2·5	2·5	625,000	11·16
6. Trautwine. . .	1·69	1·14	250,028	4·46
7. Nystrom . . .	1·0	1·24	87,500	1·56
8. Wellington . .	1·38	1·09	60,460	1·07
Formula proposed by the Author. . .	1·48	1·04	56,000	1·0

In support of the formula, the Author submits that comparison with the observations referred to confirms calculations made by it; also that certain piles driven to sustain the vibratory stress of a bridge by a ram of 1,200 lbs. weight falling 20 feet with a penetration of 0·75 inch sustained 18 tons each, and by the formula the load would exceed 17 tons. The construction of the expression has been derived from extensive observations in every variety of practice; in soil, quick-sand, boulders, submerged crib-work, and stone-filling; in water and under the embarrassing conditions of a deep trench, when the fall of the ram was necessarily arrested far above the designed bearing-level of the piles.

In a recent case, in consequence of the foundation of a building of eleven stories being 7 feet below that of two exceptionally heavy adjacent structures of ten stories each, the walls of which were laid on a concrete bed over wet sand, piling was stopped at distances of 7 feet from the adjoining walls, and the voids were bridged with plate-iron cantilever beams. The result as verified by bench-marks has proved the efficiency of the piling, which was spaced and driven in accordance with the formula submitted.

As regards the distance from centre to centre at which piles should be driven: In sand or small gravel they may be driven at 2 feet intervals, but in saturated sand and earth, or in silt, a greater distance is required, piles driven in such materials are liable to be disturbed in their position by the driving of larger piles near them. In practice, to determine the supporting-power of a range of piles, it is proper to reduce the result obtained by the formula to meet such contingencies as negligence in driving and superintendence, the frequently unobserved splitting or crushing of a pile on a stone or boulder, and the presence of quicksand.

(*Paper No. 2681.*)

(*Abridged.*)

"Concrete-Work on the Calañas-Tharsis Railway, Spain."

By Archibald Miles Anderson, Assoc. M. Inst. C.E.

The Calañas-Tharsis Railway, 18¾ miles in length, was constructed to connect the mines at Calañas with those at Tharsis, so that mineral from the former could be brought on to the Tharsis and Odiel Railway belonging to the same Company, and be thus conveyed to the port of shipment. The gauge of these railways is 4 feet.

The special feature of the railway under consideration was the extensive application of cement concrete in the construction of viaducts, bridges and culverts, and the use of rubble concrete for the piers of viaducts of considerable height. The designs of these viaducts and bridges were duplicated as far as practicable, which greatly simplified and facilitated the work, and reduced its cost.

The largest viaduct on the line best illustrates the system of construction adopted throughout (Figs. 1, 2 and 3, Plate 9). The piers were built by means of boxes of pitch-pine planking 3 inches thick, each box being about 4½ feet deep. The boxes (Figs. 4 and 5) varied in size to suit the batter of the piers, and were numbered from the top downwards; and there were always sufficient boxes in use to work on three piers and one abutment at a time. As all the permanent timber for planking the viaducts and bridges had to be of pitch-pine, 3 inches thick, it was used first for making the boxes, and afterwards as planking for the viaducts and bridges; and it was found that the cement smearing had acted as a preventative against splitting through exposure to the sun during the time it was used in the boxes.

A triangular corner-piece, fixed on inside, formed the chamfer of the corners of the piers. A bar of wood was inserted across the top of the box as a tie to prevent the side-planks from bulging when the boxes were being filled; and this bar was built in with the concrete, the connection being cut away when the boxes were taken down. The binding-straps on the sides of the boxes were

made to project 3 inches above the level of the top of the box, and to come 3 inches short of the bottom, so that the top of one box clasped the bottom of another, and kept the lower part of the latter from bulging, which secured a flush face from top to bottom of the pier.

After the excavation had been taken out, and the space was filled up level with rubble concrete, the exact height of the pier from the top of the foundation was ascertained in order to find how many of the series of boxes would be required in the pier; any part height of a box at the bottom was formed with temporary planking, and was filled with concrete before putting on the boxes in succession, only one being filled at a time. The higher boxes were filled by 2-ton steam-cranes with 60-foot jibs, erected in convenient positions, and at sufficient elevation to sweep as much of the work as possible. When filling the higher boxes of high piers, the top of the concrete was levelled off 1 foot below the top of the box, the upper planks forming a protection to the workmen engaged in filling; and the deficiency was made up in filling the next box. Although there were only sets of boxes for the construction of three piers, other piers could be begun before those were completed, as the lower boxes could be taken away, the batter on the piers preventing the boxes above them from slipping, and the corners of the underside of the lowest box left on being further supported by upright planks from the ground.

The concrete was composed of 1 part of cement to 5 parts of clean ballast or gravel, the ballast consisting of hard broken stones and coarse-grained sand. The rubble concrete consisted of irregular-sized rubble stones, set and packed solid all round with concrete. A bed of concrete, 6 inches thick, was put in before each course of rubble stones was laid; the stones were so enveloped with concrete that no one of them was nearer than 2 inches to another, and they were firmly beaten down and afterwards carefully rammed and probed all round with a trowel. Rubble stones were placed next to the boarding with their diagonals at right-angles to the face; and no rubble stone was placed nearer to the boarding on exposed faces than 2 inches, which space was filled with concrete carefully worked with long trowels so as to leave a good face. In finishing off a box, these stones were left projecting upwards (Fig. 4), so as to bond in with the concrete of the next box. The top box was filled with rubble concrete to within 2 feet of the top, the remaining space being made up with concrete alone, to constitute a bearing-block.

As regards the materials, gravel mixed with sand could not usually be obtained, and ballast had to be made with 4 parts of broken stones and 2 parts of sand, forming the required bulk of 5 parts when mixed together dry, 1 part being lost in the interstices. When this ballast, with 1 part of cement, was made into concrete and put in position without ramming, it occupied a bulk of 5 parts, and, when rammed, occupied merely 4 parts, *i.e.*, equal to the bulk of the broken stones. The Author considers that, in estimating concrete work, no reliance should be placed in obtaining more bulk than that of the broken stones. The percentage of rubble in the rubble concrete varied very much with the size of the stones used; in most cases they were flat and thin, which increased the number of concrete layers in comparison with thicker stones, reducing the amount of rubble, the highest ratio obtained being 37 per cent., and the average about 30 per cent. Where large pieces of rubble can be obtained, the percentage might be double this, and consequently the work would be much cheaper.

Eight viaducts were constructed in this way, of five to twelve spans each. The stability of the piers was proved by a serious accident which occurred in May 1887, during the erection of the girders of the largest viaduct. Twenty-four earth-work wagons broke away about 2½ miles off, rushed down a gradient of 1 in 60, and, passing the siding-points contiguous to the bridge, were precipitated over it between piers Nos. 6 and 8, carrying away the temporarily placed ironwork of spans 7 and 8, and breaking two main girders and a few of the cross-girders. Though pier No. 7 received nearly the entire shock of the falling wagons and girders, it was uninjured, with the exception of a few dents.

The piers of the smaller viaducts or bridges, with two to three spans, were constructed in a similar manner, and the arches of bridges and culverts, exceeding 5 feet in span, were built of rubble-concrete; but the small single-span bridges and culverts were made of rubble masonry laid in lime mortar. Owing to the care required in turning the rubble-concrete arches, these were executed by masons, 2 inches at least of fine concrete being placed next the soffit and between the rubble-stones, flat-bedded stones being selected for this work, radiating to the centre of the arch. Although a rubble-concrete arch may generally be considered monolithic in a sense, the Author thinks it advisable to construct it on the arch principle, as the safety of the work is ensured in the event of its cracking. Such arches, moreover, should be built as quickly as possible, to minimise the number of joinings; and not only should the arch be carried up on both sides simultaneously, but

great care should be taken that the work is left off, at the end of the day, at a regular distance throughout from the springing.

The coping and parapets of the viaducts and bridges were made of concrete blocks set in cement; and the coping of the wing-walls and the heads of the culverts was formed of concrete *in situ*, 3 inches thick. Rock foundations were always obtained for the viaducts, bridges and culverts, at a moderate depth below the surface of the ground.

The curved form of the main girders in the large viaducts was due to their having been originally parts of the iron-ribbed centering used for the Putney Bridge, London, additions having been made afterwards to suit the requirements of a girder (Figs. 6 and 7). The launching-machine was a simple cantilever, consisting of two wings built up with 12-inch pitch-pine way-beams, carried on two flat-iron wagons and back-weighted with pig-iron, a winch being fixed at the loaded end for the lifting power transmitted through the purchase-blocks at the other end (Fig. 1). As the distance between the tackle-block at the end of the machine and the rails was not great enough to allow the girders to be lifted in an upright position, they were raised lying sideways, and were run forward and launched on the piers in this manner, after which the slings were adjusted and the girder was righted and placed in its proper position. As the time occupied in this work by the locomotive was, at the outside, only about an hour, arrangements were always made to have the launching done either at meal-times, or so as to suit the work the locomotive was principally engaged upon. Immediately after the main girders were placed in position, the cross-girders were put on; and then the way-beams and rails were laid temporarily for the next day's launching. This system of launching was only carried out at the largest viaduct and two others, these being the only cases where a locomotive could be conveniently obtained. In the other large viaducts, the girders were put on in different ways; in some cases, where the bed of the river was level and the piers not high, the girders were, in dry weather, brought forward on trollies, when they were lifted into place with a 4-ton steam-crane travelling along the bed of the river on rails. In other cases, where the piers were high, or where no cranes could be had, the girders were skidded across one at a time, using the timber way-beams with rails temporarily spiked on for this purpose.

The excavation for the line amounted to 347,700 cubic yards; and, in addition to the viaducts and larger bridges referred to, there are thirteen smaller bridges, one hundred and forty-eight culverts,

three stations, six level-crossings, eight road diversions, 33,080 lineal yards of permanent way, dry-wall fencing along about a mile of the line, signals at the junction and other minor works. The work was carried out in two years through a very broken country, with no proper roads for the transport of material, most of which had to be effected with donkeys.

The railway was laid and the plans and specifications were prepared by Mr. John Strain, M. Inst. C.E., with the intention of letting the construction of the works to a contractor; but after tenders had been obtained, it was considered advisable to carry out the works departmentally under the direction of the engineer. The Author acted as resident engineer and took the management of the construction of the works described.

The Paper is illustrated by five sheets of tracings, which have been used in the preparation of Plate 9.

(*Paper No. 2664.*)

(*Abridged.*)

"The Anglo-Chilian Nitrate Railway, Tocopilla to Toco."

By Gilbert Macintyre Hunter, Assoc. M. Inst. C.E.

Tocopilla is situated in Algodon Bay, about 700 miles north of Valparaiso. There is little level ground on the coast of the northern part of Chili. A range of mountains, *Serrania de la Costa*, runs parallel to the coast, rising to a height of 2,000 to 3,000 feet; and immense quantities of débris have been carried at some remote time from these mountains and ravines, forming great banks sloping to the sea. Sometimes these banks are continued almost to high-water mark, and then descend with a steep bluff. Between this range of mountains and a second range, there is the first *pampa*, or plain; and on crossing this second range, about 30 miles distant, is the *pampa negra*, where the Caliche deposits are found, extending 30 to 40 miles from north to south, along the valley of the Rio Loa, with an average width of 4 to 6 miles. There are also copper, silver, and lead mines, and large borax deposits. Early in 1886 Mr. William Stirling, of Tacna, made a survey and report, which led to the formation in London of "The Anglo-Chilian Nitrate and Railway Company," for the construction of a railway from Tocopilla to Toco, and the erection of works for dealing with the nitrate[1] in the grounds they propose to acquire.

The most practicable route lay along the northern side of the hills bounding the ravine leading to Toco, winding in and out from one ravine to another, to reach Barriles at the level of the pampa, and crossing the cart-road at three places without interfering with it (Fig. 1, Plate 9).

The work was begun at Tocopilla in August, 1888, but little progress was made till the following January. The formation of the line reached Toco early in January, 1890, and the rail-laying was completed in March, 1890. Tocopilla Station is south

[1] "The Santa Isabel Nitrate Works, Toco, Chile," G. M. Hunter, Transactions Inst. of Engineers and Shipbuilders of Scotland, vol. 36, p. 57.

of the town, 55 feet above sea-level. The line is level for about ¼ mile to the south, where there is a reverse; then it begins to rise with a gradient of 1 in 40, gradually increasing to 1 in 25, for 4½ miles, where there is another reverse, 796 feet above sea-level (Fig. 2). The gradient of 1 in 25 is continued to Barriles Station at 17¼ miles, except where the sharp curves necessitate a flatter gradient of about 1 in 30, the line skirting the sides of the hills, passing round rocky spurs, and crossing the ravines. At Barriles the line enters upon the first pampa at an elevation of 3,285 feet. The curves of the line, with radii of 716 feet to 181 feet, forming occasionally almost complete circles, make the line to Barriles between 5 and 6 miles longer than the road. Beyond Barriles, towards Toco, the curves are comparatively few and of moderate radii, none of those on the pampa being of less radius than 716 feet. The line rises with gradients of 1 in 40 to 1 in 33 from Barriles Station to the Central Station, at 26½ miles, and thence with gradients of 1 in 66 to 1 in 50 to the summit at Ojeda, at 33¾ miles, 4,902 feet above sea-level; it then descends by easy gradients to the terminus at Santa Isabel at an elevation of 3,626 feet, 54¾ miles from Tocopilla. There is a short tunnel near Quillagua Station, through a spur of the hill, and a small wooden bridge over the road near Ojeda. Native labour was wholly employed on the work, the Chilians proving themselves to be industrious, and dexterous with the wedge, by which they excavated the rock. The northern part of Chili being rainless, it was rarely necessary to provide culverts for disposing of surface-water, and consequently all the ravines were filled with materials from the hillsides. In some cuttings, exceedingly hard rock was encountered; while in others it was soft, and was completely pulverized by blasting.

The gauge of the railway is 3½ feet, which, in the Author's opinion, is the dimension best adapted for a rugged country. It is much less expensive in earthworks, bridges and rolling-stock than is the standard gauge, while it admits of the use of equally wide rolling-stock. The cuttings are 10 feet wide, and the embankments 9 feet wide at formation-level, with slopes of 1½ to 1; while rock cuttings stand at ¼ to 1. The flat-bottomed rails weigh 40 lbs. per lineal yard, and are spiked to native oak sleepers, 8 inches by 4½ inches, varying between 7 and 9 feet in length, 2,420 in one mile. The rails as far as Barriles have alternating joints, which saved a great amount of labour and expense, owing to the very numerous curves. In spite of sole-plates and of double-spiking the outer rail in the curves, a heavy

train coming down the steep gradient and entering a sharp curve, widened the gauge, pressing out the spikes. Clip tie-rods, Fig. 3, were therefore introduced, four to each rail on all curves of less than 300 feet radius; these were made of flat iron, $2\frac{1}{2}$ inches by $\frac{1}{4}$ inch, in two pieces, bolted in the centre for ease in fixing, the outer ends being turned over the outer flange of the rail (Fig. 5). They have proved of great service in maintaining the gauge of the line, are easily applied, and do not weaken the rail. The weight of the permanent way, including sleepers, is 170·45 lbs. per lineal yard. The cost of maintenance is £6 8*s.* per mile per month.

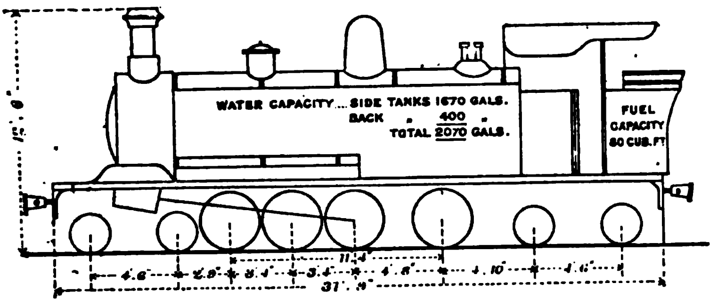
The site of Tocopilla station was in former ages covered by the sea, and has been subsequently raised to its present level (55 feet) by volcanic agency, as indicated by the frequent beds of marine shells encountered in the excavations, which were so consolidated that blasting was necessary. The traffic on the railway is controlled by telegraph, and there is also telephonic communication between Toco and Tocopilla.

Pier.—The iron pier is constructed of 5-inch steel piles, with a superstructure of plate-girders; cross girders, 18 inches deep, are placed on cast-iron caps on the top of the piles, with 15-inch rail-bearing girders between each row of the cross girders. The pier is 400 feet long and 42 feet wide, with a recess, 23 feet wide and 30 feet deep, at the extreme end for berthing launches (Figs. 4 and 5, Plate 9). There are four rows of piles to each bay, spaced 15 feet apart longitudinally and 13 feet transversely; while at low-water level there are horizontal struts to each pile, formed of two channel-bars back to back, and vertically the piles are braced with $1\frac{3}{8}$-inch tie-rods. Below low water, all the piles at the extreme end are braced, as also are three bays near the middle of the pier. The pier is 13 feet above high water, the rise of tide being 4 feet; and at the extreme end there is a depth of 26 feet at low water.

The foreshore at the site of the pier consists of hard rock with a fairly uniform surface, having a slope seawards of 1 in 13. Holes were, accordingly, jumped in the rock in order to fix the pile ends. "Jumpers" of 6-inch faces were used, with temporary clamps having four handles; they were lifted by four men, given a quarter-turn, and dropped. When the solid rock was reached, a man stood over the top of the "jumper," and assisted with a blow from a sledge-hammer at each turn. A hole to receive a pile was thus prepared in one to three hours. Most of the piles had to be placed by divers, as, although the piles were lowered from sheer-legs

exactly over the hole with ½-inch clearance all round, they were inserted with difficulty, and had to be turned round and assisted by blows from a sledge-hammer until the solid rock was reached. After a row was placed, the channel-bar struts were fixed, the level was taken, and the pile was cut off by a special cutter, in which the tools worked diagonally, one cutting V-shape and the other square. By this means the piles were cut in twenty-five minutes, instead of three to four hours, which was the time occupied by the original cutters. After a time, the men could complete two-and-a-half to three bays per week. The cast-iron screws sent out were only used in the first three bays, which are on shore. Longitudinal sleepers, 12 inches by 8 inches, are laid on the rail-bearing girders, to which the rails are spiked; and 4-inch decking is laid between, with ½-inch spaces. Walings, 12 inches by

Fig. 6.



12 inches, and fenders, 12 inches by 8 inches, are placed along each side to low-water; and two flights of steps are placed at the extreme end, on the outer sides of the recess, giving access to boats at high- or low-water (Figs. 4 and 5). Owing to the heavy surf which breaks on the coast during June and July, the erection of the pier was frequently attended with considerable danger, both to the men and to the completed part of the work; and on two occasions the temporary staging was completely wrecked.

The hydraulic plant includes a 3-ton crane with a radius of 20½ feet, and two 30-cwt. cranes with radii of 19 feet, having a lift of 30 feet. The accumulator and pumps are at the end of the pier. The pumping-engines have 10-inch cylinders with a 15-inch stroke; and the accumulator is 10 inches in diameter with a 10-foot stroke, giving a pressure of 700 lbs. per square inch. There is also a 25-ton travelling gantry crane capable of lifting a weight from a launch in the recess, and carrying it the full length of the pier. The erection of the pier occupied ten months.

Rolling-Stock.—The locomotives, of a special type to suit the heavy gradients and sharp curves, are eight-wheel-coupled tank-engines, with leading and trailing bogies, and outside cylinders 17 inches in diameter and 21 inches stroke; the heating-surface is 975 square feet, the grate-area is 16 square feet, and the steam-pressure is 160 lbs. per square inch; their weight when loaded is 51¾ tons (*Fig. 6*). Taking the mean pressure of the steam in the cylinders at 120 lbs. per square inch, the tractive force is 18,916 lbs. These engines draw four, and sometimes five loaded wagons of coal and general merchandise, weighing 80 to 90 tons, up the 17¼ miles incline of 1 in 25. From Barriles to the summit they draw ten to twelve cars, weighing 180 to over 200 tons.

The wagons are of the low-sided platform, double-bogie type, 22 feet long and 6⅔ feet wide, with sides 9 inches high, and hold 15 tons. Several of the wagons are fitted with sides 30 inches high for carrying coal. The covered goods-wagons are 30 feet long. All the rolling-stock is furnished with Turton central buffers and draw-rods, and with automatic vacuum-brakes. Without these brakes the running of loaded trains would be very dangerous; for, besides the heavy gradients, the curves are so numerous that a train is often on two, and sometimes on three, curves at one time; while the high banks of the cuttings on either side prevent the engine-driver from seeing the train or controlling the actions of the brakesmen. The brake-blocks are of cast-iron.

Buildings.—All the stations, workshops, houses, &c., owing to the want of other materials in the locality, and especially to the frequency of severe earthquakes, were constructed of Oregon pine framing covered with galvanized corrugated iron. The outer walls of all the buildings were painted and coated with a species of fine white broken shells, which improves their appearance, preserves the iron, and is easily cleaned by a jet of water from a hose.

The Author acted as Assistant Resident Engineer during the construction of the greater part of the work, and on the retirement of Mr. William Stirling, became the Resident Engineer.

The Paper is illustrated by several drawings which have been reproduced in Plate 9 and in the text, and by a statement of the cost of the works, which is presented in the Appendix.

APPENDIX.

Cost of the Works.

Note.—The cost can be only approximately given, as the rate of exchange ranged, during the three years of construction, from 29*d.* to 22*d.* The cost of the machinery is given less freight, insurance and other incidental expenses.

	$	£
Forming the line	767,000	
Laying the permanent-way .	253,000	
Stations, buildings, &c. . .	200,000	
	$1,220,000	
At, say, 24*d.* exchange		122,000
Permanent-way materials		35,307
Rolling-stock, including erection at Tocopilla		29,320
Pier with hydraulic plant		12,577
Machine-tools, &c., for workshops . . .		9,600
Telegraph-wire, instruments, &c. . . .		600
		£209,404

(*Paper No. 2728.*)

"La Guaira Harbour Works."

By William Charles Punchard and James Lennox Houston, Assoc. MM. Inst. C.E.

La Guaira, the principal sea-port of the Republic of Venezuela, is 23 miles distant by railway from Caracas, the capital, from which it is separated by a high spur of the Andes, and has a population of about 9000. In common with the principal countries of South America, Venezuela became a republic in 1826; and after a succession of revolutions, a few years of peace and settled government has led to the construction of important public works, among which is the harbour at La Guaira. This port is devoid of natural advantages, but forms the only practicable sea communication for the capital. Traffic was formerly carried on at small jetties partially protected by the remains of a wall of large stones, built about 1860, and soon destroyed; but shipment at these surf jetties was liable to constant interruption from the heavy seas which prevail. After several schemes had been proposed, the Venezuelan Government entered, in 1885, into an arrangement with Messrs. Punchard, McTaggart, Lowther & Co., of London, for the construction of a breakwater, 2,050 feet in length, protecting a water-area of about 90 acres, with an average depth of 30 feet; 3,100 feet of quays; 18 acres of reclaimed land; together with railways, warehouses, cranes, and other accessories.

Physiographical Conditions.

The roadstead of La Guaira is exposed from W.N.W. by North round to East. Although out of the hurricane zone, the effects of these storms in the Antilles are frequently felt on this coast. The terrible Martinique cyclone of August, 1891, the actual wind effect of which was confined to a very limited area, was felt strongly at La Guaira about thirty hours after its occurrence. The seas which generally prevail are those of the Atlantic, slightly modified, however, by two projecting belts of islands, the lesser

Antilles with Porto Rico and Haiti, and the Leeward Islands from Curaçao to Margarita, which lie respectively about 500 and 100 miles north of the coast. The sea-bottom, for about 10 miles from the shore, is comparatively flat, not much exceeding 150 fathoms in depth. In the next 70 miles, the bottom falls steeply to the line of the inner chain of islands already mentioned, immediately beyond which the full depth of the Caribbean Sea, 2,800 fathoms, is attained. The configuration of the sea-bottom, accordingly, accounts for the size and intensity of the waves which break on this coast in a storm. The heaviest seas come in from almost due north, ordinary heavy seas being from the north-east. The prevailing winds are the north-east trades, which blow with the greatest regularity from February to July, but are often experienced during other months of the year. There is virtually no rise and fall of tide, the slight occasional movement of less than 2 feet, which occurs principally in the winter months, is doubtless due to causes which affect the equatorial current. This current causes a steady flow along this coast from east to west, although, in the autumn months, occasional contrary in-shore currents of considerable force, are experienced, sometimes strongly affecting the sea-bottom. The heaviest storms (*mares de leva*) are usually unaccompanied by wind, being the after results of hurricanes and gales in the North Caribbean Sea and the Atlantic Ocean. They take the form of a heavy ground-swell, and, in severe cases, of long rollers coming at intervals in groups of three or four. The most severe storm of this kind experienced during the construction of the works described occurred in December, 1887. The rollers on that occasion began to show in about 7 fathoms of water, forming long unbroken lines widely separated, many of them more than 2 miles in length, gradually rising, and apparently increasing in velocity as they approached the shore. Heavy seas of a different class are common from February to May, being the results of a ground-swell combined with strong north-east winds, producing breaking seas. Columns of spray, 100 to 150 feet high, springing up from behind the breakwater, in about 15 to 20 feet of water, may frequently be seen in these months. The intensity of a storm at La Guaira harbour could always be gauged by the distance along the breakwater at which the worst seas broke. Storms of the last-mentioned description, however, are not a source of danger; and, with a north-east trade blowing, no serious effects need be apprehended. More or less ground-swell almost always prevails, and makes works of construction very difficult. Its phenomena are apparently independent of winds or waves of any ordinary

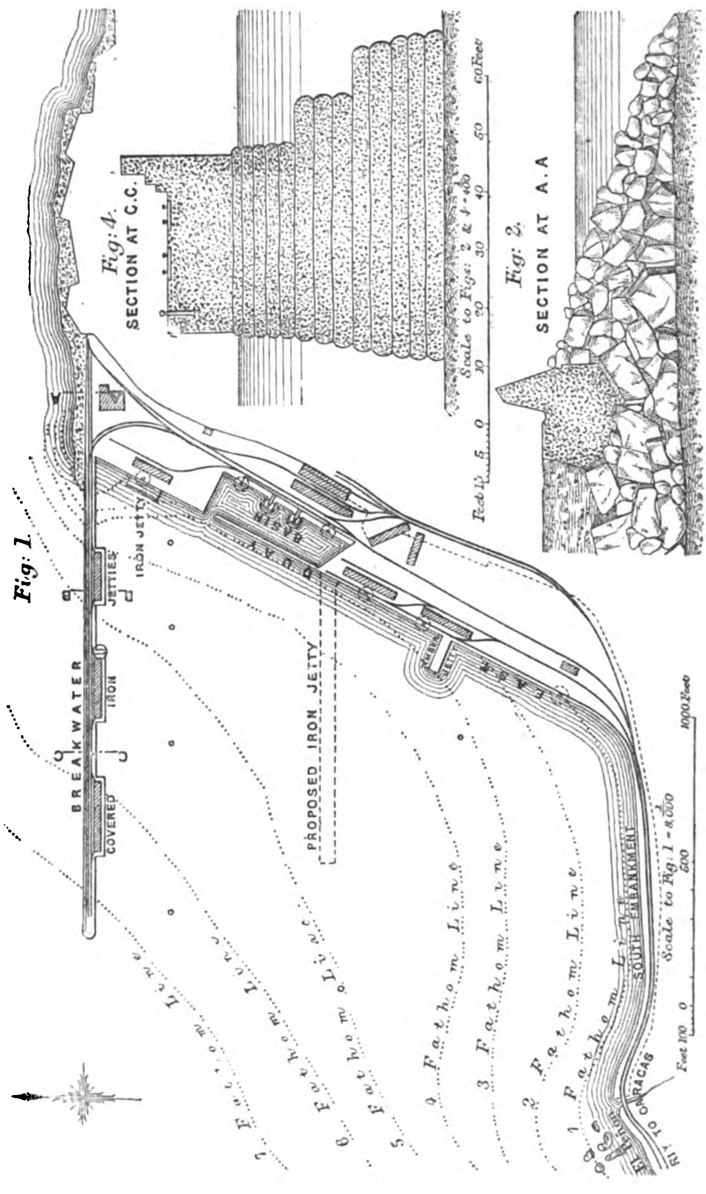


Fig: 1.
BREAKWATER
COVERED
IRON
JETTIES
IRON JETTY
QUAY
BASIN
PROPOSED IRON JETTY
JETTY
SOUTH EMBANKMENT
7 Fathom Line
6 Fathom Line
5 Fathom Line
4 Fathom Line
3 Fathom Line
2 Fathom Line
1 Fathom Line
RY. TO CARACAS
Scale to Fig: 1 = 1/8,000
Feet 100 0 500 1000 Feet
Fig: 4.
SECTION AT C.C.
Scale to Figs: 2 & 4 = 1/400
Feet 10 5 0 10 20 30 40 50 60 Feet
Fig: 2.
SECTION AT A.A

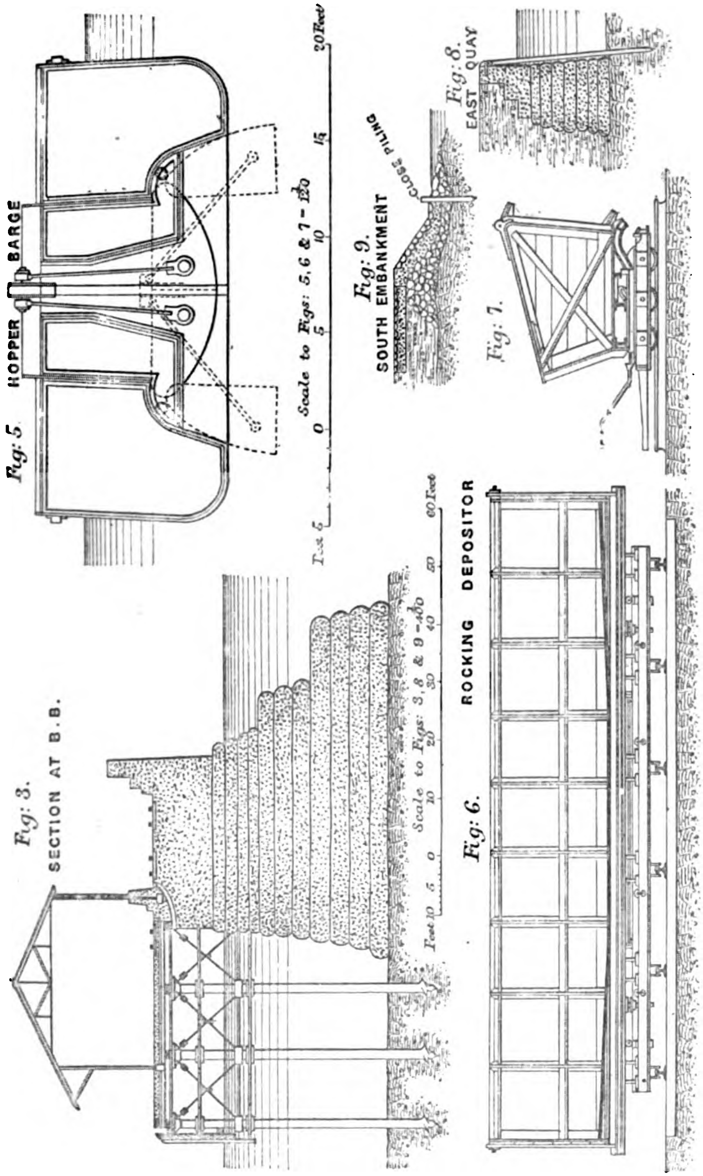


Fig: 3.
SECTION AT B.B.
Scale to Figs: 3, 8 & 9 - 1/480
Feet 10 5 0 10 20 30 40 50 60 Feet
Fig: 5
HOPPER BARGE
Scale to Figs: 5, 6 & 7 - 1/120
Feet 5 0 5 10 15 20 Feet
Fig: 6.
ROCKING DEPOSITOR
Fig: 7.
Fig: 8.
EAST QUAY
Fig: 9.
SOUTH EMBANKMENT
CLOSE PILING

kind. In the inner basin, perfectly sheltered after the works had been completed, vertical pulsations of 2 feet or more often occurred, sufficient to break mooring-lines and to prevent discharging.

Description of the Works.

The limits of exposure being almost wholly from east to north, a straight breakwater running due east and west was considered the most economical and effective line of sea defence; although, from the peculiar violence of the ground-swell, the harbour is somewhat less tranquil than was anticipated (*Fig. 1*). The design and methods of construction adopted were ruled by local conditions to a greater extent than is usual. No suitable stone was readily available, the crust of the mountain in the neighbourhood being a tough but softish clay slate. The shore, 2 miles east and 4 miles west, has considerable superficial deposits of good shingle, constantly affected, however, by the action of the sea. There was no stacking-ground near the site of the works, operations being commenced from one of the main streets of the town.

For the breakwater foundations (*Figs. 2, 3* and *4*) large bags of concrete were used. A rubble base was impracticable, not only owing to the lack of stone, but because this material could not be relied on to retain a proper slope at a reasonable depth. Experiments showed that stones weighing several hundredweight were moved in 20 feet of water; whilst in a *mar de leva*, the movement extends to a depth of not less than 40 feet.

The superstructure was a more difficult problem. The reef, formed of bags of concrete weighing about 160 tons each, dropped from a hopper barge (*Fig. 5*), could not with safety be brought up higher than 10 feet below the water-level. With regard to block-work, no land within 2 miles was available for a yard, and shingle could only be procured at distant points; whilst divers could seldom stand on the surface of the submerged reef. Any form of concrete-in-mass inside frames was impossible under water. The bag-work was, accordingly, continued up to the water-level by a rocking-depositor (*Figs. 6* and *7*), depositing tiers of sacks, and carried forward on rails on the top of its own work. The rocking-depositor was used successfully throughout; and, although the apparatus was liable to be carried away by seas, and was somewhat expensive in the quantity of jute required, as well as not giving a perfectly tight structure near the water-level, the Authors believe it to have proved, under the special circumstances,

a most useful and almost indispensable invention.[1] The breakwater was built from the water-line to the finished level, in mass-concrete in strong jute-lined frames (*Figs. 3* and *4*).

The construction of the main quay-wall proceeded simultaneously with that of the breakwater; and the site was naturally imperfectly sheltered (*Figs. 1* and *8*). An attempt was made to build this wall wholly in mass-concrete, inside jute-lined frames supported by strongly-braced piles. This system, however, had to be abandoned, as the constant suck and drag of the ground-swell caused a continual movement of the staging, frequently breaking shutters and piles, and at best leaving huge holes in the face of the work, especially round the piles. Finally, staging-piles were driven, 8 feet apart, to the gauge of the wall. A couple of planks, resting on the bottom, were then spiked on each side by divers, the immediate effect of which was that the ground-current scoured away all loose material, lower planks being added as this operation proceeded. Frequently 3 or 4 feet of sand were removed in this way. A small type of rocking-depositor, carrying bags of 8 or 9 cubic yards, and running forward on the staging, was employed, the planking being brought up so as to give a fairly full and straight face, not attainable by unsupported bags. This bag-work was stopped about 3 feet below the water-level, the remainder of the wall being finished in mass-concrete in the usual manner (*Fig. 8*). This proved an economical and excellent wall, and permitted of very rapid progress.

As there is hardly any level ground in La Guaira, it was necessary to reclaim all the land requisite for quay space and other purposes. On account of the depth of water over the site, and the difficulties of access, this reclamation was very expensive in regard to the area obtained. The trade to be accommodated is varied in character; and although the 150,000 tons per annum may appear comparatively insignificant, yet the fiscal regulations, the large number of vessels employed, and other local conditions, required more elaborate and extensive arrangements than those of most ports with a similar trade. A considerable coast-traffic is carried on in numerous small sailing and steam craft. Much of this merchandise is bought, sold, and weighed on the spot before its removal, and as the quay (6½ feet above the water-level) was found to be inconveniently high, half-height platforms were provided at the basin and adjoining quays where this department is situated.

Ocean sailing-ships are discharged principally by lighters,

[1] Minutes of Proceedings Inst. C.E., vol. cxiii. p. 23.

which again are unloaded in the basin or alongside the quay directly into wagons by steam-cranes. Steamers are discharged and loaded principally at the breakwater jetties. In a harbour of this kind, open jetties are much to be preferred to quay-walls. Nearly all the jetties and main discharging-places are roofed, an almost indispensable provision against sun and rain in a tropical country, being as well advantageous with perishable goods, enabling wagons to be loaded or discharged under cover. All the warehouses have raised cement floors, with wide outside platforms covered by projecting eaves; they are iron structures, carried on piles in the new ground, and covered with galvanized iron. Objections having been raised by the Customs Authorities to the heat of the store-rooms, canvas ceilings and screens were provided. It was found, however, that by proper ventilation, comparative coolness could be obtained; and the warehouses were ultimately provided with open-grated panels under the eaves, running from end to end, a continuous ridge louvre-ventilator, and, inside, open-work doors which secured the buildings without excluding air.

Construction of the Breakwater.

The shore part of the breakwater, known as the "sea-wall," was commenced in December, 1885. An attempt made to build this in mass-concrete proved abortive, except for a short length in shallow water. A small primitive form of the rocking-depositor was finally employed; and a light wall of 12-ton bag-work was run out to the angle formed by the quay and breakwater. The quay wall was also commenced, and a considerable area of land reclaimed, on which workshops were erected.

The storms of the winter of 1886–7 severely damaged the sea wall and the "root" of the breakwater, which had also been commenced. Breaches were repaired by tipping large quantities of boulders, which, with the débris of the fallen bag-blocks, formed a rough sea slope of 3 or 4 to 1. This was capped by a heavy concrete coping; and the construction of the breakwater, the main quay-wall, and the embankment was proceeded with. The "root" of the breakwater, which was encumbered by masses of fallen concrete and sand, was rebuilt in mass-concrete with considerable difficulty (*Fig. 2*). An outer screen of bags, similar to those of the sea-wall, was first built as a temporary shelter on the sea side, outside the line of the work. Across the end, a row of 12-inch sheet-piling was driven, the space inside was cleared of sand, and mass-concrete was deposited. A 60-ton rocking-depositor was then erected, with a steam-mixer feeding directly into it; and the

wall, 26 feet thick, capped with mass concrete 6½ feet above the water-level, was carried forward about 213 feet from the "root." In the meantime, two concrete-depositing hopper-barges were being constructed for foundation work; a mixing-station had been prepared on the inner basin, and a considerable length of quay and area of ground had been formed; and cement-sheds, workshops, and a central shingle deposit had been provided.

The storm of the 3rd and 4th of December, 1887, already referred to, rendered a reconsideration of the design imperative, to meet forces far stronger than had been anticipated. This abnormal storm occurred immediately after a severe hurricane in the neighbourhood of San Domingo. It gave warning the previous day by a single big wave, which rose out of an apparently smooth sea. During the whole course of the storm (forty-eight hours at full intensity) there was no wind, and the only appearance of movement outside the line of broken water consisted of huge flat depressions. Nearly the whole of the "sea-wall" was destroyed, being reduced to a flat slope of stones and débris. The portion of the breakwater constructed was also cut completely away, down to about 12 feet below the water-level. The "root," built in mass concrete, was the only part remaining, but was considerably damaged on the sea side. About 100 feet of the east quay, near the entrance to the basin, was also destroyed, mainly by being undermined by the immense quantities of water rushing over and behind it; and part of it sank 6 feet or 7 feet before collapsing. About an acre of reclaimed land was washed away; and great loss was occasioned to plant, staging, mixing-stations, and buildings. Masses of concrete, some weighing 40 tons, were thrown up on the quay-level, and were then moved 20 or 30 feet. The suction of the receding waves laid bare the bottom in 15 or 16 feet of water; and a large bank of sand, which had been forming behind the finished portion, was almost entirely removed, the depth there being increased 6 to 9 feet by the storm. The quantity of concrete employed in the breakwater was increased about 70 per cent., the quay-level being raised 5½ feet.

Foundation Work.

The barges, built by the Thames Iron Works Company, had hoppers 48 feet long by 9 feet wide, and 6 feet deep (*Fig. 5*). The average load was 88 cubic yards. They were fitted with pontoon doors, so as to be nearly self-closing; but the constant jarring caused by releasing the bags shook the riveting and

plating so much as to make it impossible to keep them watertight. The loaded barges were towed out by a small tug-boat, and were moored to three buoys in the required position, previously marked by floats. The fore-and-aft alignment was made by semaphores on shore, and at a signal both catches were released. Soundings were taken before and after the deposit of each bag, and frequent examination was made by diving. Very few breakages occurred, and generally only when the filling of a bag had been accidentally delayed, and setting had commenced in the lower part of the mass. The whole operation of filling and placing a bag did not occupy longer than an hour-and-a-quarter; the lacing was done on the way out; and the doors were closed, and a fresh sack was placed on the return journey. The shore-men were kept occupied in bringing forward materials, the storage-capacity of the mixer-hoppers being about 60 cubic yards. The work was brought up gradually to the full height, a length of not less than 100 yards being in progress at one time; and no symptoms of settlement or failure were at any time observed. The system admits of great accuracy and regularity of work; hollows can be made up, and a fairly level and even platform is obtained. This wall reached the exceptional height of 35 feet at the extremity of the breakwater.

The mixing arrangements were very simple: shingle brought in trains was tipped from a raised gantry, both rough and fine being thrown together to ensure a suitable mixture. Special side-tipping wagons, each of 54 cubic feet capacity, were loaded at the foot of the shingle deposit, care being taken in filling to ensure suitable proportions of sand and stone. These wagons were pushed, three at a time, up a steep approach to the mixing-platform, and were tipped into the three hoppers feeding the mixers. Cement was supplied by Decauville wagons, each of 9 cubic feet capacity, loaded in the adjoining shed, and pulled up a steep gantry by an endless chain. A load of cement was tipped simultaneously with each wagon of shingle from a somewhat higher level. These materials were then introduced into the mixing-cylinders. Water was supplied from elevated tanks fed by a small centrifugal pump, a stop-cock opposite each cylinder controlling the supply of water from a main. The mixers, designed by Mr. W. C. Punchard, were 5-foot cylinders with deep spiral blades, and proved efficient in working; the three mixers, pump, and cement-hoist were driven by a 10-HP. Robey engine. The mixed concrete was carefully trimmed in the sacks, special care being observed that fine strong material formed the outside of the block. Five bags were often

turned out in a day, but the average number was four, the output of concrete being ruled by the quantity of shingle available.

Superstructure.

The lower part of the superstructure was brought up by the rocking-depositor, in bags averaging 36 cubic yards capacity. The concrete for this was prepared at a mixer-station near the root of the breakwater, filling directly into 2-yard pans on trollies, which were pushed by a small locomotive to the face, where a 5-ton crane with a 45-foot jib, lifted and swung round the pan, and tipped the material into the box of the depositor lined with a sack. When four tiers of bags, or about 42 feet forward, had been brought just above the water-level, a strong framing was erected, and the wall was finished in mass rubble-concrete deposited principally by the crane. The quantity of concrete employed in the superstructure averaged 31 cubic yards per foot run, and the average progress was slightly over 100 lineal feet per month. In some months as much as 130 feet were constructed. The upper and lower bags of each tier were made of concrete gauged about 4½ to 1, the remainder averaging 6 to 1. The proportions, however, were constantly varied to suit the conditions of the weather, position in the structure, and quality of the shingle employed. The rocking-depositor was occasionally carried away by the sea, although strongly secured, and had to be recovered in 30 to 40 feet depth of water. This involved its being taken to pieces by divers; and so expert did the men become, that on one occasion the apparatus was at work again 36 hours after it was swept away. This work, as well as taking heavy steam-cranes to pieces in deep water, was rendered possible by the system of diving employed.

System of Diving.

The ordinary helmet was used without the dress. It rested on a circular shoulder-cushion, and was secured by a cord passing from back to front through the fork of the legs. A few turns of light chain round the middle were usually found to be necessary to keep the diver down. He was dressed in rough flannel, and was able to move and work with a facility impossible to a diver encumbered with the usual dress and weights, and with much less risk, as in an emergency he could free himself instantly. This system is only possible in the tropics, and where the water is not much colder than 68° F. No special training is required, so that the most suitable workman can be

selected for this particular service. He can work a full day under water, at a trifle over ordinary wages. The inner basin, intended at first merely to facilitate construction, was fitted out for coasting-craft, lighters, and boat-service.

As the site of the south quay-wall proved too exposed for a quay, a sloping embankment was constructed, pitched with stone in cement and protected at the toe by close piling, *Fig. 9*, steps and slip-ways being provided for fishing-boats and for the landing of timber. The proposed use of the breakwater as a deep-water quay having been found impracticable, three iron jetties were provided (two with cast-iron, and one with wrought-iron piles), designed for the heaviest class of traffic, strutted solidly against the breakwater, *Figs. 1* and *3*. These jetties are covered by sheds, closed on the sea side and the ends exposed to the weather, and have concrete floors over heavy trough-plates, excepting the outer 10 feet, which are covered by a 4-inch pitch-pine decking over concrete.

Traffic has been in operation in the harbour for three years with complete success. The works have withstood severe storms with little damage, beyond the failure of about 40 feet of the breakwater parapet in a heavy storm in October 1892.

The most notable points in the work are: 1st. Rapidity of execution, the work having been virtually re-commenced in July 1888, when the first bag of the breakwater foundation was dropped, and completed in July 1891—a maximum quantity of 13,000 cubic yards of concrete having been laid in one month. 2nd. The simplicity of the methods of construction employed, and the application of concrete bag-work in new ways with success. 3rd. The liberal use of the best machinery and appliances. 4th. The application of the "rocking-depositor," designed by Mr. Punchard, and the "helmet" system of diving.

The works were carried out by the Authors, for the contractors. The consulting engineers were the late Mr. H. Lee Smith, M. Inst. C.E., and, subsequently, Mr. A. E. Carey, M. Inst. C.E. The resident engineer during the principal part of the construction was Mr. H. F. Ross, Assoc. M. Inst. C.E. The chief assistant engineer under the Authors was Mr. H. W. Prince, Assoc. M. Inst. C.E., to whom they are indebted for assistance in preparing this account of the works. The total cost of the complete work, including railways, harbour-plant, cranes, etc., was £980,000—the cost of the breakwater being about £280 per lineal foot.

The Paper is accompanied by six drawings, from which the *Figs.* in the text have been prepared.

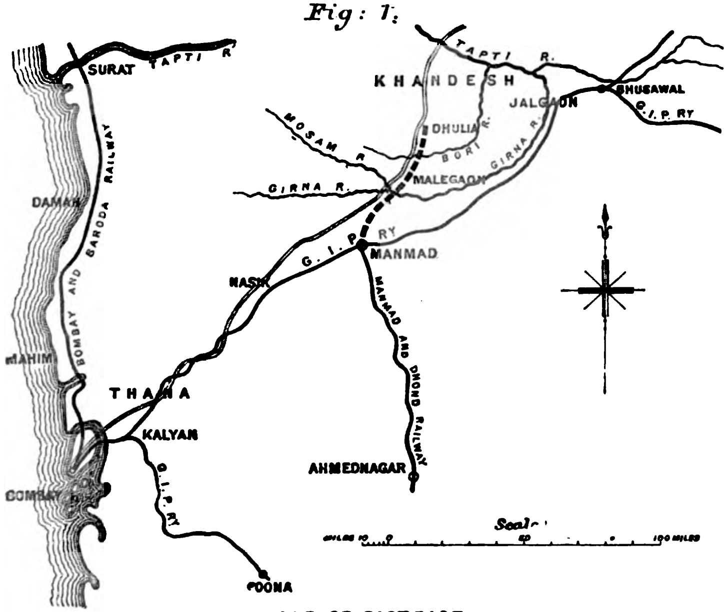
(*Paper No. 2754.*)

"The Survey of the Manmad-Dhulia Railway, India."

By Francis Robert Johnson, Assoc. M. Inst. C.E.

The line described lies between Manmad Junction and Dhulia, the principal city of Khandesh, which is the centre of a large trade with the neighbouring Tapti valley (*Fig. 1*). The distance by road from Manmad to Dhulia is 54 miles, and by the route

Fig: 1.



MAP OF DISTRICT.

surveyed, 55·35 miles. The field-work was commenced on the 13th November, 1892, and terminated on the 30th April, 1893. The cost of the survey did not exceed Rs.300 per mile.

The staff consisted of the Author and four native surveyors and levellers, with thirty assistants. The instruments used were one

6-inch and one 5-inch theodolite, one 16-inch level, four 12-inch levels, and three prismatic compasses.

Astronomical Observations.—At Manmad in November, 1892, the variation of the compass was obtained by observed west elongations of Ursa Minoris, and by azimuths from Capella in the prime vertical. The mean variation of the long-bar needle attached to the 6-inch transit theodolite was thus found to be 1° 42′ 50″ E. Transits of α Cassiopeiæ, β Andromedæ, Achernar and the Polar Star making slightly different errors of the watch, the true north line was corrected forty seconds of arc, making the variation 1° 43′ 30″. The latitude of the Author's camp at Manmad, 400 feet south of the zero point of the survey, was found by a mean of observations to be 20° 14′ 23″ N.; and 20° 14′ 27″ was adopted as the latitude of the starting-point of the traverse. The true bearing of line No. 1, parallel to the down main line of the G. I. P. Railway north of Manmad, was found by traverse from the true north line to be 69° 38′ east of north. At Dhulia, in March and April, 1893, towards the close of the survey, the variation of the compass was found from observed west elongations of α Ursæ Minoris (Polar Star) to be 1° 21′ E., and was corroborated within a minute of arc by altitude observations on Capella in the west. Star transits across the true north line as found above gave good results; and 1° 21′ E. was, therefore, adopted as the variation at Dhulia. The latitude of the Author's camp, 4,100 feet south of the centre of the River Panjhra at the west crossing, was found from a mean of observations on α^2 Geminorum (Castor) to be 20° 53′ 30″ N.; and the corrected latitude for the centre of the river was taken at 20° 54′ 10″. The angular difference of latitude between the starting-point of the traverse and the centre of the River Panjhra, west crossing, was therefore 0° 39′ 43.″ The angular difference as computed from the traverse was 0° 39′ 47″. The northing by the traverse to the centre of the west crossing of the Panjhra river was computed to be 240,838 feet; and as no difference could be detected when plotting the traverse on the 1-inch to a mile topographical survey map, the chaining was considered to have been satisfactory.

A traverse was run from west line No. 60, near Dhulia, to the true north line at the Author's camp there. The bearing of line No. 60 with reference to true north, Manmad, as deduced from the line traverse, was 329° 20′; and this made the bearing of the true north line, as staked out at Dhulia, 358° 49′, the difference being 1° 11′. Convergency of the meridian accounted for 0° 7′ of this; and the remainder, 1° 4′, was the angular error between Manmad

and Dhulia. The Author supposes that most of the error was introduced on the very rough rocky ground of the Laling range, where in many places it was impossible to drive pegs, and was very difficult to set the instrument up properly. Acting on this idea, the error was divided amongst the angles on the range for the calculation of the traverse, to be plotted on the 1-inch map.

Reconnaissance and Alignment.—For preliminary surveys over easy ground, lines were run with the aid of poles and field-glasses in the most suitable directions, assisted by the 1-inch map showing the main features of the country. These lines were roughly chained, and small pegs were driven at every 500 feet. Taped offsets, or paces, were measured to all important obstacles; a rough pencil sketch was made at every large nullah, with the aid of a few offsets and bearings, and the most suitable crossing-places were noted. A 3½-inch compass on a stand was used to take the bearings of the lines, and to roughly fix villages and other landmarks beyond the reach of offsets; and 5 to 6 miles could be got over in a day. The work was plotted at night to 600 feet to an inch; and the most suitable direction for the line was laid down to be set out next day. Had the Author to do this work again, he would use a plane-table instead of the compass, as best for a sketch survey where accuracy is not so important as a good general idea of the positions of obstacles to good alignment, marked down as the work proceeds—avoiding plotting at night, and the errors of the compass due to the abundance of magnetite in the volcanic rocks of this part of India. Over the ghâts and difficult ground, the theodolite was used. After the Author had aligned and set out the first mile from the junction to the north side of the Panjhra river, he moved his camp 5 miles, and worked down the first ghât, between 7 and 10 miles, with the theodolite forward; roughly staking out a line with pegs at 3 to 10 chains, and taking horizontal and vertical angles (*Fig. 2*). The plan and section thus surveyed were then plotted, and possible improvements were noted; and in four days a line had been set out the whole length of the ghât, with only seven curves, aggregating 5,700 lineal feet, and having a maximum radius of 1,910 feet. In the meantime, the head surveyor was taking a trial section between the Panjhra river and the ghât; which two days later was completed from Manmad to 10½ miles. The gradient obtained on the ghât was 1 in 100 on the straight, compensated for the curves to 1 in 133·33; giving two cuttings each about a mile long and 15 to 20 feet deep, and an embankment about 1¼ mile long and the same depth. By the 22nd of December, the

line had been set out a little beyond the River Girna opposite Malegaon (*Fig. 2*).

The Author then went forward to examine the second range of hills at 25½ miles, where the ghât was very short and steep, and the hills rose so abruptly from the plain that a considerable bank and cutting were unavoidable. A plan and section were taken here also by the theodolite; and by the 1st of January, 1893, the line had been staked out to the summit. The remaining portion, to 35 miles (Jhodga), was completed by the 18th. The Author then moved his camp to the 42nd mile (Arvi) to reconnoitre the Laling range, extending east and west for miles, with peaks from 1,600 to 2,200 feet above the sea, the general surface at the 40th mile, where the ascent commences, being about 1,220 feet above sea-level. The road crosses the summit of the range at 1,463 feet

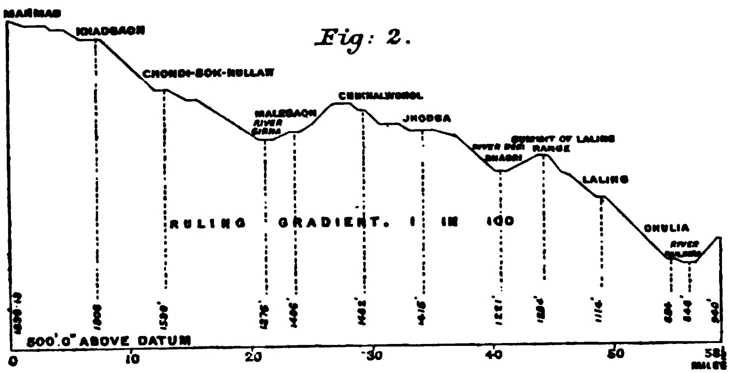


Fig: 2.

above sea-level; and at Dhulia, 10 miles distant, the level of the bridge over the Panjhra river was 846 feet, so that the fall was about 1 in 85. Two stations on the level had to be provided in this distance, and the ground fell 265 feet in the first 2 miles, or about 1 in 40; and as the ground was very rough, and extremely heavy works would have been necessary, the Author considered a line at this point out of the question. Another pass, however, was discovered, about 1¾ mile to the east of the road, considerably lower and less rough than near the road-crossing; but the ground for some miles was government reserved forest, and the jungle considerably impeded the work. There was not much difficulty with the gradients on the southern slope, which was surveyed for 2 miles; but the only means of descending the northern slope was by crossing the large nullah at the 46th mile, as near the road as possible. A suitable ravine was found near the summit,

though the ground was exceedingly steep and rocky, down which the angles for plan and section were taken with the theodolite for 4 miles to the north. There was a fall from the summit of 188 feet to the north, and of 40 feet to the south. Eighteen cross-sections were taken to the north and south of the summit, with an Abney hand-level, at the most important places, the inclination being in many cases as steep as 1 in 4. A trial line was then laid down, along which a rough section was plotted from the levels on the cross sections; this showed that, with a 60-feet bank over the large nullah, the ruling gradient of 1 in 100 compensated for curves could be obtained without very heavy work, considering the nature of the ground. The curves did not exceed 1,637 feet in radius, with the exception of a curve across the large nullah of 1,146 feet radius, where the line was deflected 90°. The setting out of this section, 2 miles south of the summit to 4 miles north of it (between the 42nd and the 48th mile), was completed by the 1st of February. For closing the gap between Jhodga, at the 35th mile, and Arvi at the 42nd mile, the 1-inch map was enlarged to 2 inches to the mile, and the traverse for the lines already set out was laid down on it. The intervening country was examined, and a good crossing for the River Bori was decided on; and in five days the setting out of this portion was completed.

At the 48th mile, near the foot of the ghât, it was just possible to obtain 3,000 feet of level for a station, after which an almost continuous gradient of 1 in 100 to 1 in 133 was required to Dhulia, with the exception of a short length of 1 in 160 at 54½ miles. In Dhulia three sites were surveyed for a station, and the east one was adopted as being the most advantageous for further extension towards Amalner and Jalgaon. Two routes were surveyed to the top of the plateau (at 58½ miles) north of Dhulia, to make sure that a future extension in the direction of the Tapti valley was so far possible without very heavy works. A gradient of 1 in 100, compensated for curves, was obtained; and the east route gave the best results as regards bridging, and the west as regards gradients.

The whole field-work, completed by the middle of April, comprised the main survey, an alternative survey of 6½ miles, near Dhulia, and an approximate alternative survey of 14 miles from Jhodga to the Chaligan road, making a total of 79 miles. At Manmad, a survey was made about 1½ mile long and ½ to ¾ mile wide, and was plotted to 100 feet to an inch for the station plan. The calculated error in the traverse for the survey was less than 1 foot.

Setting out.—After the ground had been reconnoitered, the line was set out straight from the last peg by natural objects or poles placed during the reconnaissance. The tangent-point of a curve was fixed when possible at an even number of chains, the tangent-length for the proposed angle was calculated, the distance was chained, and an intersection peg was driven. On bad ground, like the Laling range, lines were first ranged out between various fixed points to the intersections, and the angles were taken afterwards. The maximum length set out in one day was 3 miles. Every 1,000-feet peg was numbered, and had a nail driven in it exactly in the line; but the 100-feet pegs were merely placed in line by the theodolite, and were then driven.

The following was the form of record adopted :—

INTERSECTION No. 19, AT ABOUT 83,700 FEET.

1. Bearing line No. 19 . . .	in relation to true north, Manmad .	21° 0′
2. „ „ No. 20 (observed)		353° 0′
3. Inside angle (observed as check)		152° 0′
4. Turn to		Left.
5. Angle of divergence (calculated)		28° 0′
6. Radius 2° curve		2,865 feet.
7. Apex distance		714¼ „
8. Apex to curve		87¾ „
9. Calculated length of curve		1,400 „
10. Chainage P. C.		83,000 „
11. „ P. T.		84,400 „
12. Increased length of curve		1,400 „

A field survey-book was also kept to record the distances at which all nullahs, roads, &c., crossed the centre line, the direction of skew, if any, and to note width of nullahs, the appearance of the beds, direction of flow, and whether it would be necessary to have a boring made. The nature of the lands, whether dry or wet, cultivated, waste, or reserved forests, was also noted, together with a brief description of the geological features.

Spirit Levelling.—The Author checked the levels himself, as native levellers compare their books and manipulate the figures; and most of the bench-marks and all the 1,000-feet pegs were touched on — the best day's work amounting to rather over 7½ miles, including the whole of the Laling range. From a G. T. S. bench-mark at Manmad, no other was available as far as Malegaon, 23 miles away, where the Author's error was 1·33 foot. Probably most of this was due to a difference in the G. T. S. levels, as there was an error at Manmad of 1·033 foot between the levels carried along the G. I. P. Railway, and those brought from Dhond

along the State Railway, which was adjusted; whilst the levels to Malegaon were brought from the Nandgaon station of the G. I. P. Railway. Part of the error may have been also due to working in a meridional direction, one staff being always more illuminated than the other. The error was adjusted by altering all the 1,000-feet peg levels in the ratio of 0·01 foot per peg over one hundred and twenty pegs, leaving an error of 0·13 foot. These 1,000-feet peg levels were taken as standards; and where any considerable difference existed, the leveller's book was adjusted to them. From Malegaon to Dhulia, G. T. S. bench-marks were plentiful; but the Author carried his check to the end, and one hundred and forty-two bench-marks were established.

Theodolite Levelling by Angles of Inclination.—The headings adopted for the vertical columns of the survey- and the level-book were as follows:—

1. Station number (on peg)	2
2. Bearing from last station	5° 0′
3. Cardinal direction	N.E.
4. Measured length from last station	600 feet.
5. Fore angle from last station	88° 40′
6. E or D for elevation or depression	D
7. Rise	..
8. Fall	13·96 feet.
9. Reduced levels	283·02 feet.
10. Distance	1,250 feet.
11. Remarks:—300 feet from station 1, a nullah, 6 feet by 2 feet deep.	

No attempt was made to take back-angles as a check. Poles, having a narrow black ring painted on a white ground in the centre of the fifth foot from the bottom, were used in place of levelling-staves. One of these poles was used to measure the height of the instrument at each station, and another was held at the station, to which the fore angle was to be taken. The 6 inches on each side of the black ring could be subdivided by the eye with sufficient accuracy; and the height always varied between 4 feet and 5 feet. At the Laling range, the only place where errors were checked in detail by the spirit-level, the method proved accurate under very unfavourable conditions; and the Author is convinced that the theodolite is the best instrument for rapidly exploring rough ground, and taking a section by angles of inclination, which gives all the information required, in the shortest possible time, with the extra advantage that horizontal angles can be taken at the same time.

COMPARISON BETWEEN LEVELLING BY THEODOLITE AND BY SPIRIT-LEVEL.

Laling Range.

By angles of inclination.

	Feet.
South peg, No. 23	260·64
North peg, No. 35	97·71
Difference in level	162·93

By spirit-level.

South peg, No. 23	1,267·71
North peg, No. 35	1,106·17
Difference in level	161·54

Thus the difference, over $5\frac{1}{4}$ miles of rocky ground, was only 1·39 foot. The summit level was 1,302 feet, so the rise from the south was 34 feet, and the fall to the north 196 feet. The rises and falls were taken from Lloyd Gurden's traverse tables. It was unnecessary, for the trial section, to reduce the measured length to the horizontal for plotting. Deep ravines were passed over, instead of taking an angle down and up; and the widths only were noted, a few vertical offsets being taken with the tape.

Large Rivers.—Cross sections were taken of rivers having 200 feet or more of waterway, for discharge calculations, &c. These were the Sok nullah, 250 feet; the Girna river, 1,200 feet; the Bori river, 300 feet; and the Panjhra river, 800 feet wide. Five cross-sections had to be taken at each river, namely, one mile and half-a-mile above and below the crossing, and at the crossing. The ordinary and highest flood-levels, and the ordinary low-water level had to be ascertained for each cross section.

The velocity was calculated by Kutter's formula. The mean of all the cross-sections up to the highest flood-level was taken for the flood area, and half of the total fall from the first to the fifth cross sections for the mean fall per mile. The drainage-areas were taken from the 4 miles to an inch revenue survey map, and were used, together with the discharge, to calculate the corresponding coefficient for Dickens' formula, the mean being 2,570. The drainage-areas of all nullahs with water-ways exceeding 65 feet in width, requiring one or more clear spans of 40 feet, were then taken out from the 1-inch map, and the probable flood-discharge was calculated by Dickens' formula introducing a coefficient of 3,000. As all the nullahs had a considerable fall, and their beds were of hard trap rock, velocities of 15 to 25 feet per second

were used in conjunction with the available average height obtainable at each large bridge (over 40 feet span) to determine the necessary waterway.

Computing and Plotting the Traverses.—All the plotting was done by northings, southings, eastings, and westings; and two traverses were calculated for the entire length of the line. The first was reduced to true north for plotting on the 1-inch map, on which it was possible, by this method, to show every line run on the survey, and therefore the intersection of every curve. No error in distance could be detected when plotting. Between Manmad and the end of the survey, *viâ* the east (adopted) route at Dhulia, there were 62 lines, and *viâ* the west route 65 lines. The longest line was 12,254 feet, and the shortest was 709 feet in length. The traverse was computed from zero at Manmad. The second traverse was reduced to various assumed meridians for convenience of plotting. The first portion to 20° carried the plotting without a break for 26 miles, the second portion to 50° for 12 miles more, the third to 30° for 10 miles more, and the fourth to 20° again for the remaining 10½ miles, including both the east and west routes through Dhulia, and their junctions, north and south of the Panjhra river. The calculated error, on joining the lines north of the river from the bifurcation at 53 miles, was 1 foot in latitude, and 8 feet in departure. The length of this closed traverse was 5½ miles, and the width was 2 miles. A third traverse was computed for the alternative line from Jhodga, at 35½ miles, which crossed the range further east, giving lighter work and gradients, but a much longer line.

The Paper is illustrated by a tracing, containing plans and sections of the proposed line, from which the *Figs.* in the text have been prepared.

(*Paper No. 2752.*)

(*Abridged.*)

"Surface Tin-Mining in the Malay Archipelago."

By Charles Reginald Parkes, M. Inst. C.E.

Tin-mining in the Malay Islands consists of prospecting for and exploiting deposits of stream-tin derived from the degradation of stanniferous rocks by rain and floods and their subsequent decomposition, the débris being deposited in adjacent valleys and river-beds. Traces of stream-tin may be spread over large tracts of country, but paying quantities are found only in certain favoured positions. The search for these rich patches is a difficult and costly operation, since the existing streams and depressions often occur in directions different from those which originally influenced the deposition of the ore. The particles of tin oxide range in size from fine grains to pieces larger than a cubic inch, the latter being generally found in elevated positions at the foot of the mountains, and the finer mineral being on the plains and the sea coast. The large pieces require to be crushed for the removal of impurities; but the fine particles have a specific gravity of 6·9, and are of such quality that neither calcination nor any other purifying process is required. These oxides vary in colour with the locality in which they occur, and are known as black, red, and white respectively.

The district which it is proposed to open up for mining is first surveyed, and systematic boring is then undertaken to determine the exact position and the size of the tin-ore streams, careful pannings being made during these operations. The boring is, advisedly, superintended by a European engineer, who may have under his charge three to five boring-gangs, each one consisting of nine Malays and a native foreman. The boring-tubes are usually of $4\frac{5}{8}$ inches diameter inside. The strata traversed before the "*karang*," or layer of stanniferous gravel, is reached, consist generally of small stones, blue clay, and sand. Underlying the "*karang*" a stratum of compact yellow clay, locally known as "*kong*," is met with; and through this it is generally useless to

bore. If the stanniferous gravel be more than 45 feet below the surface, it is generally unprofitable to open a mine, and, even at that depth, only an unusually thick and rich deposit can be worked.

In some cases 1 *tael*, or 0·083 lb., of tin oxide per metre of depth bored is necessary for profitable working; in other cases, the rule is to require 40 *pecul*, or 5,293 lbs., of refined or smelted tin per 100 cubic metres of excavation without reference to depth. The foreman of each boring-gang is provided with a half shell of cocoa-nut for panning the gravel brought up by the drill. This half shell has a capacity of about 0·000625 cubic metre, and should contain 4 *hoen*, or 0·8 pint, of tin ore. The formula used at this stage is

$$x = \frac{g}{h} \times 0{\cdot}534,$$

where x is the number of *pecul* per 1,000 cubic metres;
g „ weight in *hoens* on average of borings;
h „ average depth of borings in metres.

The number thus obtained covers an allowance for loss and waste during working.

The shape and size of each mine depend on the width of the tin-stream and other conditions; but the excavation is generally about 100 to 200 feet square. The sides are made as nearly vertical as the ground permits, and are in many cases cut in low terraces, the faces of which are strengthened by vertical stakes wattled with matting and boughs of trees. The tin-stream is followed step by step, the overburden being deposited in pits already exhausted.

To utilize the available water-power to the greatest advantage, dams are placed across the valleys at the base of the mountains, so that the supply may be regulated as far as possible by the demand. The mines are opened in sets at convenient elevations, one above another, so that the tail-water from the upper mines may have sufficient head to be of use on reaching the lower series. With an adequate water-supply, the surface soil may be hydraulically removed to a depth of several feet, and the head of water may be further utilized in working the mine-pumps and in separating the ore from the other constituents of the gravel. The pumps used for draining the mines are either European, usually driven by steam, or Chinese, worked by water-power or by manual labour. The shallow mines are generally drained by

means of native water-wheels, constructed entirely of bamboo, and caused to rotate by coolies treading on the descending spokes. The well-known Chinese chain-pumps, which may be worked by hand to a 13-foot lift, appear to give much better results than these. But such power is only used in very small or shallow mines. Overshot water-wheels 3 to 6 feet in diameter, constructed by Chinese carpenters, are also used in these tin mines. Chinese pumps give fairly good results for lifts up to 25 feet, but beyond that depth the loss of efficiency is great; and for the larger and deeper mines steam-pumps are required, the centrifugal pump, with pipes 3 to 6 inches in diameter, being preferred. Pulsometers are valuable adjuncts in the event of accidents. Portable engines, or vertical multitubular boilers with vertical or horizontal engines, self-contained and requiring little foundation, are most suitable for this class of work.

The mining operations proper are all performed by manual labour. The ground is excavated, placed in small baskets suspended from shoulder-sticks, and carried by coolies to the surface up inclines formed of notched tree-trunks. The stanniferous gravel is dumped in heaps from which the large stones are raked out, the ore then passes on to an inclined trough, 40 feet long by 2 feet wide and 1 foot deep, placed immediately below a cascade of water. The gravel, in descending the trough, is pushed back and turned over by shovels skilfully manipulated by men standing in the stream, the lighter matter being thus gradually washed off the slope, and the heavier oxide of tin remaining. To check the velocity of the descending material, three or four bars, or weirs, are placed across the trough. The tin ore thus obtained is melted in a blast-furnace built of a mixture of blue and yellow clays containing 5 to 10 per cent. of salt. The fuel used is charcoal, and the blast is furnished by an ordinary fan or by Chinese bellows. The melted tin is run into cast-iron moulds, each having a capacity of ½ *pecul*, or 66·16 lbs. of the metal. Imperfect ingots are remelted in a pot on an ordinary stove.

Borings have been made during the present year with a view to determine the truth of traditions as to the existence of rich streams of tin-ore in the shoals on the sea-coast, but up to the present time they have not been productive of satisfactory results.

(*Paper No. 2717.*)

"A Method of Underpinning and Tunnelling under Tenements."

By JAMES YOUNG, Stud. Inst. C.E.

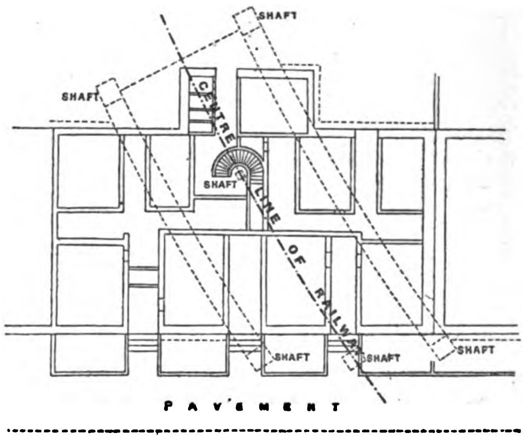
THE work described is one of the numerous operations of engineering interest which have been undertaken in carrying out the Glasgow Central Railway,[1] which connects the East and West of Glasgow, passing under the busiest thoroughfares in the city. On the Stobcross contract of this railway, the work passes under St. Vincent crescent, which consists of dwelling-houses. The tenements are three stories high, exclusive of the basement, and have a frontage of 80 feet and a depth from front to back of 42 feet to 55 feet. Each tenement is divided into six dwelling-houses, the upper floors being reached by a common stair and entry. The internal walls of the basement are shown in *Fig. 1*, which also indicates, by dotted lines, the relative position of the railway. Seventy-five yards east of the works described, the City and District Railway cuts through a gap in the crescent formed by the removal of one tenement. The level of the rails is, however, 12 feet 6 inches higher than this crossing. Another gap in the crescent would have not only seriously interfered with the general character of the crescent, but would have required very substantial buttressing of the adjacent houses. It was therefore decided to either take down and rebuild, or to pass under the property, and the contractors were asked to consider this in tendering. Various methods for underpinning were discussed. To build the side-walls and arch piece-by-piece in pits sunk from the surface, presented serious practical difficulties, especially having regard to the angle at which the railway crossed the crescent; and to have carried the underpinning of every internal wall to the formation-level was out of the question, as the cost of doing so would have exceeded the value of the tenement. Ultimately it was decided to construct a relieving-arch of concrete as an outer shell, under cover of which the internal brick arch might be built with safety.

Trial-pits showed the foundations to be in fine sand saturated

[1] *Vide* "Methods adopted in constructing the Glasgow Central Railway (Bridgeton and Trongate Contracts)," by C. D. Barker, Stud. Inst. C.E., Minutes of Proceedings Inst. C.E., vol. cxiv. p. 340.

with water and very unstable, underlaid by a bed of solid sandstone a little above the formation-level. In the first instance, to drain the ground, a heading was driven below the formation-level, so as to keep beneath the rock surface. Except in a few places where the top of the heading broke through the rock, it was unnecessary to timber the work. The heading was driven under the houses with a slight rising gradient, so as to drain both the ground in which the underpinning was to be done and that at the back of the crescent. This heading served its primary purpose admirably, and proved a most useful service-heading, having been purposely made large enough to permit the passage of small wagons. Nearly all the material excavated was conveyed

Fig. 1.



Scale, 1 inch = 40 feet.

through it to an adjacent shaft, where it was lifted by a hoist and emptied into trucks.

As soon as the sub-soil was sufficiently drained, underpinning was commenced by sinking four pits, two in front of and two behind the tenement, on the line of the side-walls (*Fig. 1*). These were carried down to the rock and were connected with the heading by small shoot-holes. From the bottom of the shafts in front, headings, 4 feet wide and 6 feet high, were driven half-way under the tenement on the line of the tunnel-walls. These headings were then carefully packed with concrete to within a few inches of the top timbers. Similar headings were then driven from the shafts behind the crescent to meet those already made. The latter also, when completed, were packed solid with concrete. Another set of headings of similar dimensions and carried out in

the same way on the top of the concrete walls already built, raised the side-walls 5 feet higher. The concrete in the second tier was not brought close to the roof-timbers throughout, but was finished so as to form the springing for the arch. The timbering of these headings was of the simplest kind, the greater part of it being left in.

Fig. 2.

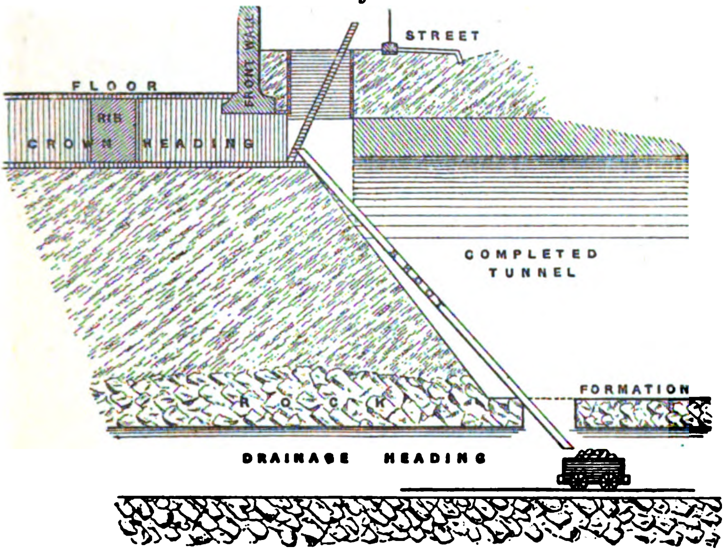
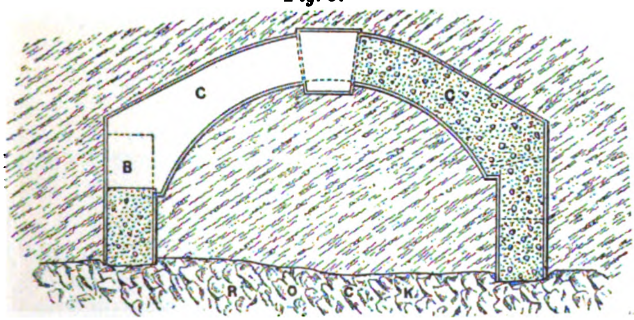


Fig. 3.

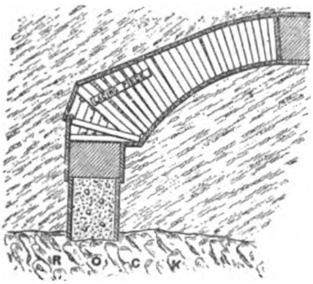


Scale, 1 inch = 16 feet.

According to the original plan, it was intended to drive on the top of this an inclined heading, forming a horizontal segment of the relieving-arch. A better plan was, however, adopted, *Fig.* 2, by which, from a small heading 4 feet high driven along the crown of the arch close under the foundations of the building, the

segments of the relieving-arch were built. The outside walls of the building having been securely lintelled, the crown-heading was driven half way through. From each side of this, cross headings CC, *Fig. 3*, were driven some distance apart down to the springings formed on the side-walls already described. These rib-headings were carefully shaped by a template in the dry sand to the radius of the arch, and each one was filled with concrete as soon as it was excavated. The intervals between these were then filled in by other similar headings similarly concreted, the irregularities in the concrete caused by the side-timbers helping to bond the ribs one to another, *Fig. 4*. The underpinning of the front of the building having been thus accomplished, the top heading was carried through and the back was underpinned in a similar manner. Finally, the crown-heading was filled with concrete and the arch was thus completed.

Fig. 4.



Scale, 1 inch = 16 feet.

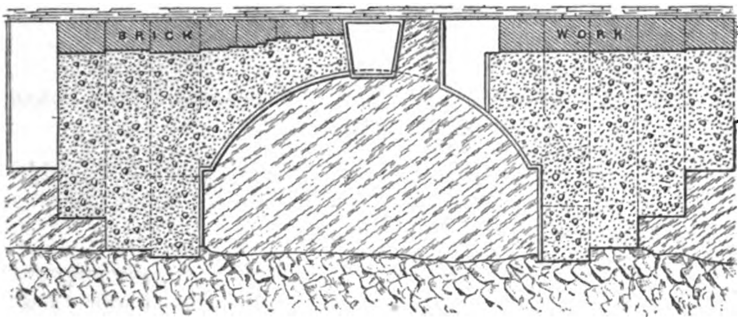
Under the outside walls "wings" from the concrete side-walls were stepped up from the rock level till they ran out at a slope of 1 to 1, *Fig. 5*, about 3 feet on the top being brickwork, which, with wedge-shaped bricks where necessary, was built tight up to the foundations. The skew crossing complicated the work somewhat, so that at some places it was necessary to sink pits about 4 feet square under the foundations, and from these pits to drive short ribs down to the springings. After each rib was concreted, the pit was also filled with concrete, and the upper half of the rib was made in the open. At other places, the ribs were made in the usual way under the walls, and the concrete wall was built in short lengths between the top of the ribs and the foundation of the building above.

All concrete was gauged 1 part of Portland cement to 6 parts of sand and broken stone. Where new and old concrete joined, the latter was brushed clean, and, where necessary, was roughened on the face. The timbering was simple in character, consisting of a roof of 3-inch timbers laid crosswise, supported by side-timbers or legs of similar scantling. All timbering was close, although the ground was at that time quite dry, for the vibration caused by the traffic in the street was apt to shake the sand out through any cranny in the sheeting. On the completion of the arch, and after the concrete had been allowed time to set,

the core or dumpling was removed and the brick lining was put in, the finished tunnel being in cross-section as shown in *Fig. 6.* The houses above were occupied during the whole period of the operations, the work being successfully completed without the building being in the least degree affected.

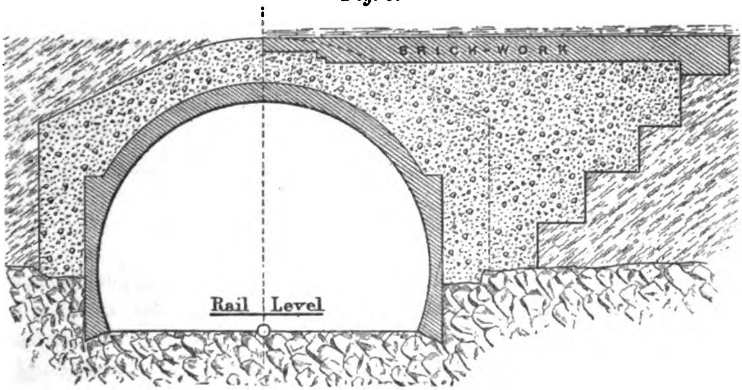
It is believed that the method adopted for accomplishing this work is novel, and is capable of extensive application in con-

Fig. 5.



Scale, 1 inch = 16 feet.

Fig. 6.



Scale, 1 inch = 16 feet

nection with railway-work in towns. Finding rock at a fairly high level was no doubt an advantage, while the water in the material, and the underpinning of a heavy stone staircase just over the crown-heading, presented grave difficulties in the execution of the work.

The communication is accompanied by a sheet of tracings from which the *Figs.* in the text have been prepared.

(*Papers No. 2600 and 2653.*)

(*Abridged.*)

"Hamilton Graving-Dock, Malta."

By CHARLES COLSON, M. Inst. C.E., and CHARLES HENRY COLSON, Assoc. M. Inst. C.E.

WHEN the extensive works at Malta dockyard, forming the subject of this Paper, were commenced, there were three docks in operation, as given in the following Table:—

Description of Dock.	Period of Construction.	Length on Blocks.		Width at Coping.	Width on Floor.		Width at Entrance.		Depth on Sill.	
		Feet.	Ins.	Feet.	Feet.	Ins.	Feet.	Ins.	Feet.	Ins.
No. 1 Dock in Dockyard Creek	1841–48	256	1	82	26	9	81	3	25	0
No. 2 Dock, continuation of No. 1	1856–62	269	7	90	38	4	73	2	25	0
No. 3 Dock, Somerset Dock in French Creek[1] . .	1865–71	427	8	104	42	6	79	11	33	11

HAMILTON DOCK.

The new works necessitated by the growing importance of Malta as a naval station, comprise:—(1) A dry dock capable of receiving almost any of the long ships of the mercantile marine available as transports or armed cruisers, as well as all naval ships; (2) Additional wharfage and appliances to accommodate two or more of the largest ironclads; (3) Factory for the repair of ships and fittings; (4) Hydraulic crane capable of lifting 160 tons;[2] and (5) Shops and stores for the repair and storage of gun mountings.

The site eventually selected for the dock, under the advice of the late Lieutenant-General Percy Smith, R.E., Director of Works,

[1] Minutes of Proceedings Inst. C.E., vol. xxxiii. p. 352.

was on the east side of French Creek, to the north of Somerset dock (Fig. 1, Plate 10), as it is close to the dockyard shops and stores, sheltered from strong winds, easy of access from the harbour without interfering with the deep-water anchorage in the creek, and sheltered from an enemy's fire by the high rock on which the town of Isola stands. This position, moreover, has enabled the pumps and engines to be duplicated with those of the Somerset dock, a valuable provision in case of a breakdown and enabling either dock to be pumped dry in half the time on an emergency. The range of spring tides at Malta is from 10 to 16 inches. The site of the dock was almost entirely covered by high, obsolete fortifications, necessitating the removal of 260,000 cubic yards of rock and rubbish down to coping level before commencing the dock excavations.

The principal dimensions of the Hamilton Dock are:—

	Ft.	Ins.
Length on floor	520	0
With the caisson in the outer stop	558	0
Width at coping	126	0
„ inside entrance	118	0
„ of entrance	94	0
„ on floor	63	0
„ between broad altars	97	0
„ of entrance at broad altar level	90	2
Depth of water on sill	35	6

The arrangement of the altars, steps and slides is shown in Figs. 1 and 2, the recesses being arched over so as to preserve the continuity of the coping.

The excavations, with the exception of the surface soil, materials and rubbish of the fortifications, and mud in the harbour, were made in soft calcareous sandstone, resembling Bath stone, and varying greatly in quality and durability. The rock was excavated by trenching and wedging, and subsequent breaking up instead of by blasting, owing to the proximity of the town of Isola, and the preference of the Maltese for their ordinary methods. The rock below the top 6-foot layer was mostly removed by travelling-cranes lifting wooden boxes with flap-doors, and iron turnover skips holding 1½ tons, most of the material being carried out to sea in hopper-barges. The rock in which the dock was formed was too soft and liable to disintegration under the influence of the atmosphere to bear the wear and tear to which the lining of the dock would be exposed, and it was, moreover, considerably fissured, the fissures varying in size from mere cracks to 8 inches in width, with occasional larger cavities, sometimes

choked up with mud or clay, and sometimes quite clear, allowing large quantities of water to pass. The overlying rock was liable sometimes to slip on the wet clay in the fissures.

In view of the difficulties encountered in the construction of the Somerset dock it was determined that the pumping should be concentrated at one or two points, and that the permanent pumps should be brought into use as soon as possible, temporary pumping arrangements being in the meantime resorted to. To test the nature of the rock, and the probable influx of water, trial-shafts were sunk in some cases to the full depth of 55 feet without meeting with water; whilst sometimes fissures were met with which were stopped with wooden wedges, and in one instance with Portland cement grout, which was forced in under water, before the sinking was continued. These shafts gave more favourable indications as to the amount of water likely to be met with than the experience of the Somerset dock warranted. The results were therefore regarded with suspicion, and as not proving the absence of large water-bearing fissures, a view which subsequently proved to be correct. The position of the dock having been finally settled, shafts, 6 feet in diameter and 55 feet deep, were sunk on the centre-line of the dock, and connected by a heading, 6 feet by 4 feet, running along the whole length of the dock, and conveying all the drainage water to the pumps; as the excavation proceeded, the shafts were joined by an open trench formed below the bottom of the excavations, which kept the work dry. These excavations revealed fresh fissures which had been missed by the heading; and a shaft, 30 feet by 23 feet, sunk at the extreme head of the dock, for collecting almost all the drainage, passed through very fissured rock, discharging large volumes of water. Eventually three No. 11 and two No. 9 pulsometers, and two 12-inch centrifugal pumps, were placed in this shaft, pulsometers being selected as requiring no expensive machinery to work them, as taking up little space, and as requiring little fixing, being easily lifted for examination or repairs, and for the rapidity with which they can be set to work—a quality which saved the works from being flooded at a critical time. On the completion of the large sump to the full depth the permanent culvert to the old well was carried out, rendering the Somerset dock pumps available for dealing with any sudden influx of water down to 40 feet below coping level. The drainage from the entrance and caisson chamber, and the leakage through the circular cofferdam outside (Plate 10, Fig. 1), was provided for by another set of pumps. The greatest quantity of water was met

with at the head and stern of the dock, and little could be done to diminish the volume to be pumped by closing the fissures till the limits of the excavation were reached.

The fissures in the main sump and permanent wells were traced to the face of the rock in the lead to Somerset dock, and were then stopped to a great extent by clearing them out, and packing them up tightly with Portland cement in small bags. A series of fissures was stopped by sinking a trench, 4 feet wide, between the dock and the wharf, to a depth below an enlargement through which most of the water passed; the fissures on the dock side of the trench were then stopped with neat Portland cement, and the trench was filled up with puddled clay and cement concrete. Many smaller fissures were closed by inserting a stone at the lower part of the fissure, with a hole through it for the escape of the water, and then cutting out the fissure to a dovetail shape back to the solid rock, and filling up with neat Portland cement, with masonry set in neat cement, or with Portland cement concrete; when this filling up was quite set, the hole was plugged, and a dove-tailed recess in front was filled with cement. Where the flow of water was too great to admit of this simple method, the water was passed through a cast-iron pipe, fitted with a screw-down valve, built into an opening made in the solid rock; the fissure was then cut back and filled up as before described, when the filling was thoroughly set, the valve was slowly closed. Fissures met with in sinking the penstock shafts were stopped either by pumping in liquid Portland cement through a pipe inserted in the fissure and blocked in with cement, or by concrete deposited under water when the rock was too much broken up to allow cement to be pumped in. In the latter case, the defective rock was taken out all round to an extra width of 1½ to 2 feet, and deep enough to clear the fissures in the shaft; after allowing the water to rise to its normal level, the cavities were filled with rich Portland cement concrete. Then, after the lapse of about fourteen days, the shaft was re-sunk through the concrete. On the completion of the excavations, the open trench along the bottom of the dock was utilized for collecting the drainage from the larger fissures extending across the floor of the dock by laying a 15-inch cast-iron pipe in 9-foot lengths, with a 4-inch branch on each length right and left alternately, in the bottom of the trench, and embedding it in Portland cement concrete, the rest of the trench being filled up simultaneously with the concrete of the floor. The fissures were then connected with the 4-inch branches by stoneware pipes laid in a small trench in the rock, and bedded in neat cement, or by narrow

trenches covered with flat stones fixed down with neat cement and concrete. The central drain was continued through the new suction culvert to the main wells. Milk of lime was frequently used for tracing the direction of a fissure, or for detecting any connection between the fissures. Many small fissures in the bottom of the dock were dealt with by cutting a very narrow dove-tailed trench as deep as possible on the line of the fissure, with a small grip along the bottom just large enough to carry off the water, which was covered with a piece of thin wood, sheet-iron, or zinc, the trench being filled up above with neat cement. Several of these small grips were collected into one channel and connected with one of the branches of the central drainage-pipe. This pipe was divided into sections by screw-down valves at suitable intervals, enabling the drainage during construction to be distributed between the pumps at the head and stern, and, on the completion of the work up to the water-level, the leakage to be cut off from each section in succession.

Lining of Dock.—The floor is Cornish granite, in courses 3 feet wide and 2 feet and 3½ feet thick alternately, bedded on Portland cement concrete (Fig. 2). The upper and lower altars, steps, and timber slides are the best local limestone backed with concrete. The broad altar and copings, and the invert, haunches, filling culvert entrance, and quoins of the entrance, are granite filled in with limestone above the haunches. The invert of the caisson groove across the entrance is limestone, and within the caisson camber Portland cement concrete. The slide stones for the caisson are granite, and the guide stones within the camber are limestone. The rock was sufficiently sound between the fissures to render it only necessary to excavate it to an adequate depth to obtain a watertight bed of concrete under the floor stones, well keyed to the rock. This keying was effected by cutting transverse dovetailed trenches, 4 feet wide by 1 foot deep, and 12 feet apart. Longitudinal trenches, 3½ feet wide and 1¾ foot deep, were also cut where practicable, about 10 feet apart; and the trenches for stopping fissures served the same purpose. These transverse grooves, with smaller horizontal connecting grooves, were continued up the sides of the excavation to the water-line, thus keying the concrete well into the rock, and avoiding a continuous straight joint between the rock and dock-lining.

Proportions of Concrete.—The mortar of the concrete in the floor and up to the first altar was composed of 1 of cement to 1½ of sand, in consequence of the numerous fissures; from the first altar to the water-level 1 of cement to 2 of sand; above the

water-level 1 of cement to 3 of sand; and elsewhere, in backing or filling, 1 of cement to 4 of sand. Experiments with mortar in the different proportions gave the following results:—

Cement.	Moderately Fine Sand.	Fresh Water.	Mortar.	Increase Per Cent. on Bulk of Sand.
1	1½	0·72	1·90	26·6
1	2	0·90	2·36	18·0
1	3	1·20	3·10	3·3
1	4	1·56	4·00	..
1	5	1·74	4·58	Decrease. 8·4

The stone used for the concrete was broken partly by machine and partly by hand, so as to pass through a ring 2½ inches in diameter. The interstices in the stone amounted to 50 per cent. when hand-picked, and 40 per cent. for stones of all sizes up to the limit, that is, broken stone as used in the work, representing the smallest proportion of mortar requisite for sound concrete. Only the hardest and best stone was used for the concrete of the floor, and up to the first altar; in the backing of the altars up to the water-level, the hardest and second quality stone was used; and above, the second quality stone was used, an admixture of softer stone from the excavations having been abandoned as unsatisfactory, as it was crushed in ramming.

A loss of 10 per cent. of mortar was at first allowed for in making the concrete, giving with 1 of cement to 1½ of sand, 1·71 cubic foot of available mortar, or sufficient for filling the interstices of 4·27 cubic feet of broken stone as used in the works; but as it was soon found that under the conditions obtaining practically no waste of mortar occurred, 4·75 cubic feet of stone were needed for utilizing the whole of the mortar; and the proportions actually adopted were:—

No.	Cement.	Sand.	Stone.	Situation of Concrete.
1	1	1·5	3·5	Interstices reckoned at 50 per cent.; over fissures and wet parts of floor.
2	1	1·52	5·0	Interstices reckoned at 40 per cent.; dry parts of floor.
3	1	2·1	6·0	Interstices reckoned at 40 per cent.; backing to altars.
4	1	3·0	8·0	Interstices reckoned at 40 per cent.; above water-level.
5	1	4·0	10·0	Non-watertight backing.

The concrete was in no case dropped from a height, but was mixed by hand as near its final position as possible, and deposited by the native system from baskets holding 1 cubic foot, in a layer about 1 foot thick, which, being well rammed formed a thoroughly homogeneous mass. Where it was impossible to mix the concrete near the level of deposit it was passed down through a shoot 10 inches by 10 inches, and turned over before being filled into baskets for deposit. A small area was preferred for the shoot, as then the concrete was not screened in falling. Tests made of the watertightness of the concrete in the different proportions, of which an account has been previously given,[1] indicated that a somewhat smaller proportion of cement might have sufficed if the number of fissures had not made it expedient to take every precaution to ensure the water-tightness of the work. The roughness of the rock obtained by the pick proved unsatisfactory for receiving the concrete, as dust and dirt accumulated in the conical holes. The rock was, therefore, dressed over quite smooth by a tool resembling a narrow adze. And, after being well wetted and scrubbed, a 2-inch bed of neat cement was spread over the whole surface. Before this cement had set the concrete was deposited in a 12-inch layer and rammed uniformly till the water appeared on the surface; and whilst it was wet, the next layer was deposited and rammed, this being repeated till the level of the shallow stones was reached, the chase for the deep stones being formed in the mass. By depositing the floor in sections, not too large to be completed in a day, horizontal joints were avoided, and, being finished in steps which were cleaned and rendered with neat cement before commencing the next section of concrete, the continuity of the concrete was insured. The floor-stones were laid on the concrete in a bed of 2 inches of neat cement, the deep stones being set first. The floor-stones were set by a steam derrick-crane spanning the 76-foot width of the floor, and serving two setters. The lower altars were set by steam travelling-cranes in the bottom; and the broad altar, upper altar, and copings were set by travelling-cranes on the quay-level. The sliding ways and guide stones in the caisson camber were set partly by steam-cranes, and partly by shears and tackle on the quay; and the stones of the entrance were set by travellers on gantries. The first floor-stone was set in September, 1888, and the last coping-stone in September, 1890, when the valves in the drainage-pipes were closed, and the pump-shafts filled up; and the caisson was placed in the entrance in

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[1] Minutes of Proceedings Inst. C.E., vol. cvii. p. 179.

May, 1891. No leakage occurred through the masonry after the closing of the drainage pipes; and the leakage from the caisson and filling culvert was only 1 gallon an hour.

CAISSON.

The rectangular sliding caisson, made of mild steel, is 40¼ feet high and 16½ feet wide, exclusive of the keel and stem timbers, and is strengthened by two watertight decks, and bracing and framing (Figs. 6 and 7, Plate 10). As the position of the entrance precludes heavy traffic passing over the caisson, the roadway deck could be placed low enough to pass under the covering of the camber, 1¾ foot below the coping, connection being made with the quay by a hinged flap. The caisson can be floated out from its normal position to the outer stop, thereby adding 38 feet to the available length of the deck. The air-chamber, 92 feet by 16¼ feet by 8⅛ feet, in the middle of the caisson, is reached through two shafts. The caisson is ballasted by concrete blocks on the floor of the air-chamber, and by water in the tanks under the roadway deck at each end. Without any ballast the caisson would float with the top of the air-chamber 2 inches above the water; but the concrete ballast more than balances the flotation, producing a normal pressure on the sliding ways of 10 to 20 tons. The water-ballast is adjusted by means of a three-way stopcock in the 4-inch pipe connecting the tanks, enabling the water to be run from one tank to the other, or one or both tanks to be emptied. The caisson can be hauled in or out of the camber in five minutes, by two steel pitch chains connected with the hydraulic hauling-gear, and exerting a pull of 30 tons on the two projecting arms of the caisson to which they are attached. The caisson is guided into the camber by the keels and granite rubbing pieces below, and by fenders and rubbing pieces above; and tilting is prevented by the adjustment of the water ballast, and by rollers on the underside of the camber girders. The caisson is stopped automatically at the end of its course into or out of the camber, and buffers are placed in the recess opposite the camber, in case of a failure of the automatic stopping gear. The maximum tensile strain on the plating of the caisson does not exceed 6½ tons per square inch, when one side of the caisson is dry, and the water is up to deck B on the other side. The keels and stems are greenheart, 10½ inches by 8 inches, and the rubbing pieces and fenders are American elm. Two sluice-valves, 3½ feet in diameter, and 1½ foot above the deck floor, furnish an auxiliary means of filling

the dock. A four-inch hand-pump serves to remove water from the air-chamber. The hauling arms can be readily moved when the caisson has to be floated out of place.

Pumping Station and Culverts.

The drainage-wells are at the head of the dock, and at the rear of the Somerset dock wells, together forming one pumping-station available for either dock. The main well, 52½ feet below coping level, was excavated in the rock, 33 feet by 10 feet by 11½ feet high, above which is the pit containing the pumps and engines, 39 feet by 17 feet and 30¼ feet deep, with its floor 20¼ feet below coping level. The whole arrangement being below the water-level and in very fissured rock, had to be lined with hard stone backed with cement concrete. The drainage-pump well, 10½ feet in diameter, and 57½ feet deep, is alongside the main well, and connected with it by a culvert, 4 feet by 2½ feet. The suction culverts connecting the new dock with the old and new wells, 6 feet by 5 feet, and 46 below coping level, terminate in a sump, 15 feet by 5¼ feet, and 6 feet deep below the dock floor, at the back of the recess at the head of the dock. The new well is also connected with the Somerset dock suction culvert by a branch culvert; and the old and new drainage wells are connected by an 18-inch pipe with a screw-down valve for cutting off communication. The discharge culvert from the new well is 13 feet by 8 feet, its bottom being 16 feet below coping, and grooves for a timber dam are provided in the valve-chamber. A culvert, 3¾ feet by 2 feet, with a fall of 1 foot in the length of the dock, connecting a sump, 4 feet by 3½ feet just inside the entrance, with the main suction culvert, serves for removing the last part of water at the entrance, for the ordinary drainage of the dock, and also for emptying the caisson-chamber when required for repairing the caisson. The filling culvert, 6 feet by 7 feet, with its invert 30 feet below coping, is situated on the west side of the entrance, and directed at a slight angle to the face of the entrance to divert the inflowing water from the face of the caisson.

In sinking the new wells, much difficulty was occasioned by fissures; and on the completion of the excavation for the main well to the full depth, the water burst up from a large cavity a few inches below the bottom, raising a large slab of rock, bringing in large quantities of mud and red clay, and flooding the works in a few minutes. The water being somewhat lowered by cutting through into the Somerset dock culvert, the fissure,

which was large enough for a diver to get into, was opened out and cleared by a diver; and on letting the water rise to sea-level, neat Portland cement, made of the consistency of mortar, was placed in the fissure in the hope that it would find its way into parts out of reach of the diver. The cavity was then filled with Portland cement concrete, let down in iron buckets with covers, which, when set, stopped the influx of water, and enabled the wells to be pumped out. An invert and lining of hard stone were eventually laid over the whole bottom of the well.

Dams.

In determining the method to be adopted for excluding the water, local conditions had to be considered. The surface of the rock was extremely irregular and generally at a sharp slope underlying a stratum of very soft mud, in some places of a considerable depth. Clay for puddle was difficult to obtain and of a very inferior quality. Piles could not be driven into the rock, the effect of driving after the surface was reached being to start the foot of the pile down the slope. There being practically no tide, the difficulty of fixing shores and walings was much augmented; a double row of piles would have tended to spread at the bottom on the sloping rock, owing to the impossibility of introducing tie-bolts below the surface of the mud. It was therefore decided to form the dams with a single row of piles, sawn on the sides in contact to insure a watertight joint. The site of the wharf wall was enclosed by two rows of piles, 27 feet apart for a length of 680 feet, with whole timber walings and shores (Fig. 4, Plate 10). To allow for sudden changes in water-level which frequently occur in Malta Harbour, and for the slight tidal oscillations when the dam was approaching completion, openings were cut in the piles, which were closed just before pumping was commenced. To obviate irregularities in driving, owing to frequent obstructions in the deeper parts with a considerable thickness of mud, grooves and tongues were formed by spiking wood fillets, 4 inches by 2 inches, to the sawn sides of the piles. This was not required on the shallower parts of the dams, and was avoided where possible, as it was found that the sawn faces made the most watertight joint. The piles had to be weighted in the deeper part of the dam till the stiffer mud near the rock was reached. The outside pressure on the inner row of piles, on the pumping out of the water, tended to displace the piles at the foot on the sloping rock before the mud inside could be removed and additional shores put

in, leading to serious leaks. This was guarded against by casting a quantity of stone debris into the dam before pumping out commenced; and on preparing for a tier of shores and walings, the stone was merely piled up in the bays till the timber was in place. The width between the two single-sheeted dams inclosing the site of the wharf wall was made much greater than required for the wall, in order to make certain of the absence of faults in the rock, tending to produce a slip under the wall, and to avoid shaking the rock by blasting close to the wall, and also because the removal of the rock in the dry was much cheaper than by blasting and dredging under water.

The circular dam in front of the entrance was formed in a series of straight bays or cants of about 20 feet long, this form being adopted in preference to a true curve, owing to the greater facility in fitting and driving the piles, and in fixing straight walings; which were important advantages where skilled labour was scarce, and in view of the fact that much work had to be done under water (Figs. 1 and 5, Plate 10). The dam was braced and strutted at the cants to single piles driven 20 feet back inside, and connected by walings. Owing to the steep slope of the rock, the piles varied greatly in length, the longest being 48 feet. In the deeper parts of the dams, the driving of the piles was sometimes arrested by large stones, timber, or masses of debris. The nature and size of the obstacle were then ascertained by borings all round the pile; and in some cases large stones were split by clearing off the mud, boring holes through the stone, and then driving down the pile. The leakage through the dam and the cost of maintenance were insignificant; but the crab "cheluria terebrans" and the worm *Teredo navalis* seriously ravaged the soft Triest timber piles, which, after trying tar, stiff clay, and iron and zinc sheeting without success, were at last vanquished by a coating of neat Portland cement grout poured through a hose into an interstice $\frac{3}{4}$ inch wide, formed by nailing boarding to wooden fillets projecting $\frac{3}{4}$ inch in front of the piles at intervals.

WHARF-WALL.

The wharfage to the north of the dock, for a length of 750 feet, has a depth of 30 feet of water alongside, affording berthage for two large ships, whilst the remaining 250 feet has a depth of 10 to 12 feet alongside. The concrete wall, deposited *in situ*, is faced in the upper part, and coped with native limestone set in

Portland cement mortar (Fig. 4, Plate 10). The concrete for the facework, 3 feet in width, was made of 1 of cement, 2 of sand, and 5 of broken stone; and for the backing, 1 of cement, 3 of sand, and 8 of broken stone. The concrete was lowered through vertical shoots, only 12 inches square, so that the mass thrown into hoppers on the top might not be screened in falling, which would occur with a large shoot placed at an angle; and the concrete was re-mixed at the bottom before being deposited. The shallow portion of the wall, for about 30 feet, was built upon the foundations of an old wall; but as the rest of the old wall had not been founded on the rock, it had to be removed by divers and a Priestman grab. Benches in the sloping rock were formed by steel cutters, fixed in a heavy cast-iron head, 9 inches in diameter and $4\frac{1}{2}$ inches deep, attached to boring-rods. These cutters were dropped upon the rock through a square guide trunk, and turned through a quadrant after each blow. Holes were thus easily cut, 12 to 14 inches in diameter, close together, to the requisite depth, any projecting portions missed by the cutters being removed by divers. The concrete for this part of the wall was then deposited *in situ* within framing under water, from boxes with flap bottoms containing one cubic yard. The mass was raised quickly in sections to the required level, to avoid horizontal joints; a groove, formed in the concrete by the timber bulkhead at the end, served to key one section into the next. The milky layer, found each morning on the surface of the concrete put in the preceding day, was brushed off before depositing fresh concrete.

Where the rock had to be excavated to give a depth of 30 feet alongside the wall for a length of 750 feet, and the depth to the rock in the line of the wall varied from over 40 feet to under 12 feet, it was important that the rock immediately in front of the wharf wall should be excavated in the dry, so that all fissures dipping outwards, endangering the stability of the wall, might be discovered and dealt with. On encountering such fissures, the rock was cut away so that the foundations might be laid below the line of the fissure; and vertical dove-tailed grooves were cut in the rock at the back at intervals of 6 to 8 feet, which, being filled with concrete in bringing up the wall, keyed the wall to the rock. Loose debris filled a depression in the rock at the deepest part, and necessitated carrying the foundations at this part considerably below the main pumps and drains; and when the rock was bared, numerous water-bearing fissures were disclosed. As pumps at a considerably lower level would have been required to dry this foundation, and as the influx of water

was increasing, and time was of the utmost importance, the foundation, for a height of 2 feet, was formed of cement-bags, two-thirds filled with concrete, laid in bonded courses and rammed into a solid mass. The upper surface of these bags was then cut away, and the surface rendered with neat cement before the wall was continued by depositing concrete *in situ*. Another fissured depression through which water issued in a strong current was too irregular and deep to fill up satisfactorily with concrete bags, a sheet of strong canvas was therefore stretched over the hole, large enough for the edges to be kept out of water, in which the concrete was deposited, filling up all the irregularities.

Some sudden blows occurred, bringing large quantities of water, with harbour mud, through fissures into the enclosed space. In one case it was found that the row of piles had crossed a fissure, but had not penetrated far into the debris with which it was filled. When the dam was pumped out, this filling proved not strong enough to withstand the hydrostatic pressure, and was blown in. To stop the leak, the mud had first to be cleared away; the fissure was then cleared out, as far as practicable, and filled up with neat Portland cement mortar, overlaid with Portland cement concrete passed through the water in closed buckets. After allowing some days for setting, the enclosure could be pumped dry again. Two slips of rock from under the dam occurred, owing to the existence of an imperceptible fissure on which the sliding took place. In the one case, the slip occurred at a part where the wall was finished, and consequently, did not cause inconvenience. In the other case, the aperture under the piles caused by the slipping away of a mass of rock 8 feet long, had to be closed inside the dam under water with concrete bags, and the cavity outside was filled up with concrete in bags at the bottom, overlaid with concrete deposited through the water. After an interval of ten days, the water within the dams was pumped out without the occurrence of any leakage.

Removal of Rock in Entrance Channel.

The removal of the rock in front of the graving-dock and wharf wall outside the dams had to be postponed till the graving-dock was finished, the caisson in place, and the wharf wall built, for fear of disturbing the dams and increasing the flow of water through the fissured rock into the works. The rock consisted of a calcareous sandstone, similar in appearance to Bath stone but

harder, practically without bedding, solid in itself, but fissured, and cracked at varying distances and directions. The bulk of rock to be removed, to a depth of 35½ feet below the water-level, was large (Fig. 8, Plate 10), whereas its area was small, so that blasting would have been tedious and inconvenient, even if the proximity of the rock to the caisson and wharf wall, and to the town of Isola, had not precluded its adoption. Added to this, time was of the utmost importance, as it was desired to have the use of the dock with the least possible delay. Breaking up the rock by Lobnitz's system of falling cutters[1] obviated these objections, and enabled the removal of the rock to be effected close after the cutter, thus greatly reducing the time of the operations as compared with blasting. The staging for the cutters was erected in the centre of a platform resting upon two barges, each 65 feet by 18½ feet and 7 feet deep, and the bulk of the rock was shattered from this central position; but for breaking the rock close up to the walls a gantry was erected at one end of the platform, from which one cutter was worked. Three 8-ton rams, or cutters, 39½ feet long, with a steel point welded in, were lifted by wire ropes working on pulleys at the top, 3 feet in diameter, and actuated by steam winding-engines, the vessel being moved by a winch pulling on chain moorings. The top guides for the cutters at platform level were fixed, but the bottom were hung in chains at the water level, and had a slight travel backwards and forwards, which absorbed the shocks of the falling cutters.

The drop given to the cutters was at first from 10 to 20 feet, but was eventually reduced to 7 feet; for though the higher drop broke up more rock, the pieces were too large for readily lifting, whereas a 7-foot drop and quicker blows produced pieces easily raised by the grabs. The cutters were kept moving to and fro over a distance of 6 to 8 feet, until a depth of 3 feet of rock had been broken up. Occasionally projecting cones of rock, 2 to 6 feet high, were formed by the cutters sliding different ways, owing to the fissures and cracks above mentioned, which often had to be blasted. The cutters should always be long enough to cut to the full depth with two guide frames in place, for the want of the upper guide to keep the cutters straight favoured the formation of these points, and materially reduced the output of the machine when working at the full depth. The average quantity cut, measured on the sections, per day of 13·84 hours, taking the whole time the machine has been at work, including stoppages,

[1] Minutes of Proceedings Inst. C.E., vol. xcii. p. 369.

was 12·1 cubic yards per chisel per day. The average number of blows per cubic yard was fourteen, and 1·93 cubic feet of rock were broken per blow. The blows given per chisel per hour, whilst the machinery was in motion, averaged 21·19, but taking the whole time the crew were on the works as working time, they averaged 12·25. Occasionally the cutters stuck in the rock, but, with one exception, they were readily extracted by moving the vessel backwards and forwards.

The $1\frac{1}{2}$-inch iron chains wore rapidly by the grinding of the ends of their links together in passing round the barrel and pulley, which should be of larger diameter. Galvanized steel-wire ropes, subsequently adopted, produced less jerk on the winches, were less dangerous in the event of breakage, were less liable to sudden rupture, and lasted much longer. Three of the cutters gave from 26,300 to 33,000 blows before needing repairs; but subsequent experience showed that they should be annealed after 20,000 blows on rock similar to that at Malta. The cost of breaking up the rock, including a proportionate charge for the cutting machinery, but exclusive of the cost of old barges, was 6*s.* $4\frac{3}{4}$*d.* per cubic yard.

The broken rock was lifted by Wild single-chain grabs,[1] worked by the steam-cranes which had been used in the dock works, placed on two old barges. Four grabs were used, made by Messrs. Stothart and Pitt, and costing £500, two of them being half-tine, and two whole-tine. The whole-tine grab was most used for lifting broken rock and stiff mud, whilst silt, soft mud, and soft crushed rock were best raised by the half-tine grab. The broken rock raised varied from sand to 5-ton blocks. The greatest quantity of broken rock lifted by one grab in a day, from 36 feet of water, was 105 tons in 134 lifts, and of stiff mud, from 8 feet of water, was 245 tons in 143 lifts. The average amount dredged per grab per day of 12·87 hours, in 53,780 lifts, and including all stoppages, was 31·66 tons of broken-up rock, or 62·93 tons of hard stony mud, scraped from the top of the rock. The lifts per day per grab averaged 109, or about 7 minutes per lift, in a depth of water of from 8 to 36 feet. The grabs got the broken rock much better when following close on the rock cutter, as in time the broken rock settled down into a compact mass more difficult to penetrate. The grabs were not affected by moderately rough weather when working in a bottom free from obstructions, but when lifting rock, or where liable to be caught, a high wave was liable to lift the barge when the grab was retained, causing a

[1] Minutes of Proceedings Inst. C.E., vol. lxxxix. p. 46.

capsize or carrying away the gear. The cost of labour, stores, and repairs per ton of broken-up rock raised was $10\frac{1}{4}d.$, and for stiff, stony mud, mostly scraped from the top of the rock, $5d.$ per ton. The weight of the material lifted was measured in barges holding 90 tons, sinking to a special mark, the tonnage of the barges having been determined by weighing with weighed material. The cost of towage out to sea is not included in the above prices, nor any charge for plant.

The Papers are illustrated by ten sheets of tracings, twenty photographs, and thirty-five sketches, from which Plate 10 has been compiled.

APPENDIXES.

APPENDIX I.

Tests of Portland Cement and Mortar.

The following experiments were carried out with cement supplied by different makers. The briquettes were broken in one of Adie's straight-lever machines fitted with an automatic apparatus for applying the load regularly. The samples were mixed by three methods—(1) by "hand"; (2) by "machine";[1] and (3) by a "screw-press" as follows:—The cement was put dry into the mould, standing in a shallow tray, and pressed down by a plunger the exact area of a briquette, the sides of the mould being made extra high to serve as guides. Sufficient water was then put into the tray to saturate the cement, and leave about 1 inch remaining, care being taken that no water fell on the top of the cement, but was all taken up by capillary attraction. After twenty-four hours the briquette was removed from the mould, and immersed in water until required for testing.

The difference between "hand" and "machine" when neat cement was used was small, the average of eighty-eight tests giving 365·67 for "hand," and 362·74 for "machine." When, however, sand and cement were mixed, the machine showed a considerable advantage, namely, 165·69 for "machine," 145·84 for "hand," and 122·62 for "press." The grains of sand appear, therefore, to be more efficiently covered with cement by the action of the machine than by hand-mixing.

Table I.—Comparative Strength of Mortars Made of Different Kinds of Sand, Mixed in the Proportion of 2 of Sand to 1 of Cement. Same Cement used Throughout. Tested at Three Months.

Number of Experiments for each Result.	Sand used.	Sizes of Grains of Sand. Passed through 64 meshes per Square Inch.	Through 64 meshes. Stopped on 256 meshes per Square Inch.	Through 256 meshes. Stopped on 625 meshes per Square Inch.	Through 625 meshes. Stopped on 2,500 meshes per Square Inch.	Through 2,500 meshes per Square Inch.
		Average Breaking-Strain per Square Inch.				
No.		Lbs.	Lbs.	Lbs.	Lbs.	Lbs.
36	Siliceous sand with a slight amount of calcareous sand	146·00	171·95	169·71	146·15	131·39
36	Calcareous sand of siftings of limestone from stone-breaker	267·63	272·71	240·56	218·98	218·26
24	Sand of equal parts of each of the above, mixed	202·38	205·81	197·75	180·06	176·19

[1] Minutes of Proceedings Inst. C.E., vol. lxxv. p. 223.

TABLE II.—COMPARATIVE STRENGTH OF NEAT CEMENT GAUGED BY "MACHINE,"[1] AND BY "HAND." TESTED AT SEVEN DAYS.

Number of Briquettes Tested for each Result.	Average Breaking-Strain per Square Inch for Neat Cement Briquettes. Gauged—	
	By "Machine."	By "Hand."
	Lbs.	Lbs.
88	362·74	365·67

TABLE III.—COMPARATIVE STRENGTH OF A MIXTURE OF 1 OF CEMENT WITH 2 OF SAND, SCREENED THROUGH 625 MESHES PER SQUARE INCH, AND STOPPED ON 2,500. GAUGED BY "HAND," "MACHINE," AND "PRESS." TESTED AT FORTY-TWO DAYS. THE SAME CEMENT USED THROUGHOUT.

Number of Briquettes Tested for each Result.	Breaking Strain per Square Inch. Briquettes Gauged by—		
	"Hand."	"Machine."	"Press."
	Lbs.	Lbs.	Lbs.
24	145·84	165·69	122·62

TABLE IV.—THE RELATION OF THE FINENESS OF PORTLAND CEMENT TO ITS STRENGTH. THE SAME CEMENT USED THROUGHOUT, WEIGHING WHEN RECEIVED 81 LBS. PER CUBIC FOOT. SPECIFIC GRAVITY 3·07 AND 7·5 PER CENT. RESIDUE LEFT ON SIEVE OF 2,500 MESHES PER SQUARE INCH. TESTED AT FORTY-TWO DAYS.

Method of Mixing.	Fineness of Cement as Tested.			
	As Received 7·50 per cent. on 2,500 Meshes.	Through 2,500 Meshes per Square Inch.	Through 3,600 Meshes per Square Inch.	Through 5,800 Meshes per Square Inch.
	Lbs.	Lbs.	Lbs.	Lbs.
Hand . .	482·96	574·44	603·77	580·73
Machine .	471·11	536·66	537·03	528·88
Press . .	483·55	507·22	522·77	499·02
Averages .	479·20	539·44	554·52	536·21

Each result given is the average of six experiments.

The above Table is given for what it is worth, but the number of experiments are not sufficient to make it reliable.

[1] Minutes of Proceedings Inst. C.E., vol. lxxv. p. 223.

TABLE V.—RELATIONS BETWEEN THE STRENGTH OF MORTAR, THE COARSENESS OF THE SAND, AND THE FINENESS OF THE CEMENT. MORTAR MIXED AS 2 OF SAND TO 1 OF CEMENT. TESTED AT FORTY-TWO DAYS.

Number of Experiments for each Result.	Size of the Grains of Sand.		Fineness of Cement as Tested.			
	Screened through.	Stopped on.	As Received.	Screened through 2,500 Meshes per Square Inch.	Through 3,600 Meshes per Square Inch.	Through 5,800 Meshes per Square Inch.
	Meshes per Square Inch.	Meshes per Square Inch.	Lbs.	Lbs.	Lbs.	Lbs.
			Breaking strain per square inch.			
12	64	..	172·77	195·36	211·10	214·11
6	64	2,500	120·36	144·81	150·46	153·92
12	2,500	..	120·54	135·55	148·05	136·66

Cement measured by volume.

TABLE VI.—RELATION OF THE FINENESS OF CEMENT TO THE STRENGTH OF MORTAR.

Size of Sand.	Fineness of Cement used.			
	As Received.	Through 2,500 Meshes per Square Inch.	Through 3,600 Meshes per Square Inch.	Through 5,800 Meshes per Square Inch.
	Lbs.	Lbs.	Lbs.	Lbs.
	Breaking strain per square inch.			
Screened through 2,500 meshes per square inch	122·22	133·70	203·25	240·36

Each result is the average of six experiments. Same weight of cement used in all the experiments.

TABLE VII.—RELATION OF THE STRENGTH OF MORTARS MADE WITH (SO-CALLED) SILICEOUS SAND TO THE AMOUNT OF CALCAREOUS SAND CONTAINED IN THEM. MIXED BY "HAND"; MIXED AS 3 OF SAND TO 1 OF CEMENT. TESTED AT TWENTY-EIGHT DAYS.

Size of Sand.	Percentage of Calcareous Sand contained in the Sample.			
	nil	Per cent. 2	Per cent. 4·22	Per cent. 8·45
	Breaking strain per square inch.			
Screened through 256 meshes and stopped on 625 meshes per square inch . . .	Lbs. 93·55	Lbs. 103·18	Lbs. 117·77	Lbs. 120·96

Each result is the average of six experiments.

The siliceous sand used was carefully prepared by soaking in sulphuric acid and water to remove all the lime possible, and then thoroughly washed to remove all traces of acid. It was then mixed with a known proportion of calcareous sand of the same gauge of grains before adding the cement. The increase in strength as the calcareous sand was added is clearly marked, and tends to show the great care that would be required in selecting standard sands for testing Portland cement to ascertain that the amount of lime was the same in each sample.

APPENDIX II.

Cost.

Construction of dock, including excavation in rock, removal of material, pumping and total cost of all plant and materials, extreme internal dimensions, 558 feet long, 126 feet wide on coping, and 41 feet deep. The price includes the excavation and removal of rock outside the caisson as far as the end of entrance horn, but not the cover to caisson camber, caisson, bollards, nor fairleads.

	£	s.	d.	
Cost per cubic yard of internal capacity, with caisson in outer stop, but not including the capacity of caisson camber	1	15	9	In each case the price includes the total cost of all timber, plant, and materials used.
Cost per foot run of dock, from outside of horn at entrance to back of coping at head	252	18	0	
Single-pile circular dam, 100 feet radius, arc 242 feet long, bracing four tiers deep, 20 feet wide; average length of piles 30 feet, extreme length 45 feet per foot run	31	18	6	
Maintenance and repairs, including sheeting with cement, divers, etc., for six years per foot run per annum	1	0	0	
Single-pile dam for wharf wall, two rows of piles, average depth of front row 31 feet, and of back row 24 feet, extreme depth 45 feet . . . per foot run	25	17	0	
Wharf wall, average depth 27½ feet, extreme depth 42 feet, including excavation of trench, 6 feet wide in rock in front of wall to a depth of 35½ feet, pumping and removal of debris . . . . per foot run	24	13	0	
Steel caisson, including hauling machinery, chains, slides, engines, and deck to caisson camber. Entrance, 94 feet wide and 41 feet deep per foot width of entrance	156	5	0	
Per superficial foot of caisson over keels	4	3	0	

APPENDIX III.

Division of Working Time of Lobnitz Rock Cutter.

Average number of hours per day, 13·84.	Per cent. of Total Time.
Machinery was actually in motion	59·44
Repairing	11·25
Shifting ship to moorings morning and evening and preparing for work	18·63
Altering working-moorings	2·41
Stopped for meals	7·04
Stopped by rough weather	1·23

Crew Required.

1 master, 4 winch-men, 3 stokers, 4 deck-hands, 1 boy, 1 watchman. —Total 14.

APPENDIX IV.

Details of Cost of Breaking up Rock with Lobnitz Cutter.

Total number of cubic yards broken up as measured from sections = 7,515.

	£	s.	d.		Per Cubic Yard. s.
Total expenditure—					
Wages	464	12	0	=	1·227
Repairs	147	3	1	=	0·391
Water-service	42	5	3	=	0·113
Fuel	211	16	7	=	0·563
Engine-stores	48	16	2	=	0·130
Ship „	10	6	6	=	0·027
Total cost per cubic yard					2·451
Total cost of machinery, chains, anchors, spare gear, &c.	5,958	4	2		
Deduct one quarter, estimated selling-value on completion	1,489	11	0		
	4,468	13	2		
Add cost of fitting up old barges and erecting machinery	462	4	10		
Total cost	4,930	18	0		

Divided over 25,000 cubic yards of rock, the approximate cost is $\dfrac{£4{,}930\ 18s.\ 0d.}{25{,}000} = 3\cdot947$

	6·398

Total cost of breaking up rock ready for lifting = 6*s.* 4¾*d.* per cubic yard, exclusive of the value of old barges or of old timber used in fitting up.

OBITUARY.

JOHN PHILIP CORTLANDT ANDERSON, son of the late Major Philip Cortlandt Anderson, of the 64th Bengal Infantry, was born in India on the 24th of June, 1831. His career as an engineer commenced in December, 1850, when he entered the service of the Public Works Department of the Government of India. For seven years he acted as personal assistant to Lieutenant (now General Sir) Alexander Taylor, R.E., Superintendent of the military road then under construction from Lahore to Peshawar, of whose office Mr. Anderson had charge. In September, 1857, he was sent on special duty to Jalandhar and Umballa, to audit some official accounts which had fallen into arrears and confusion.

Promoted to Fourth Grade Executive Engineer in 1858, Mr. Anderson was in the following August placed in charge of No. 8 Division of the Grand Trunk Road, which appointment he held until January, 1870. During that time he constructed the metalled and bridged road between Lahore and Ferozpur, some 48 miles in length—a work which involved the contraction to about 2 miles of the width of the Sutlej river bed at the latter place by means of spurs and embankments; built a bridge of three arches, each of 40 feet span, on well-foundations over the West Beyne Nullah river; and repaired the well-foundations and renewed the superstructure of four of the seven arches, each of 30 feet span, over the Budha Sutlej river. Every year he renewed from 40 to 60 miles of road and erected several bungalows on the Grand Trunk Road. The bulk of the plant used in constructing the bridge of boats over the Bias river was also made up by him. All these works were projected, designed and estimated for by him, and were executed under his supervision.

Mr. Anderson was next placed in charge, in 1870, of the Umballa Division, which post he held until his retirement from the Department in 1879. His principal occupation during that period was the design and construction of military barracks. He was also in charge for a time of the waterworks at Simla and Lahore. His steady and untiring application to his duties won him on more than one occasion the thanks of the Government of India. At the

time of his retirement he had risen through the various grades to the rank of Superintending Engineer.

On leaving India Mr. Anderson ceased to practise as an engineer and spent the remainder of his life in retirement in England. His death, which was very sudden, occurred on the 14th of August, 1893. On the morning of that day he started, apparently in good health, to bathe at Woolacombe Sands, near Ilfracombe. On entering the water he fell forward helpless; his fellow-bathers hastened to his assistance, but before he could be brought to dry ground life was extinct. His medical adviser certified that death was due to heart disease.

Mr. Anderson was shrewd, hard-working and energetic, a skilled accountant and a good man of business. He was elected an Associate of the Institution on the 6th of December, 1870, and was transferred to the class of Member on the 4th of November, 1879.

SIR GEORGE BERKLEY, K.C.M.G., was born on the 26th of April, 1821, and was educated at a private school at Hampstead. At the early age of fourteen he was removed from school and apprenticed to Messrs. Samuda Brothers, the well-known marine engineers. After going through the shops he assisted in preparing designs for railways worked on the atmospheric system, to which that firm was then giving considerable attention.[1] About the year 1840 he had the good fortune to be received—through his brother James,[2] who was then acting as secretary and assistant to that gentleman—into the office of Robert Stephenson[3] at No. 24 Great George Street, Westminster.

Shortly before this appointment George Berkley, who had been in somewhat delicate health, was advised to try country air. For a time he lived at Bishop Stortford, and while there took advantage of the immediate neighbourhood of some repairing shops on the Eastern Counties system to make a number of connected experiments and observations on the working of locomotives under varied conditions. To this work he was prompted by an irresistible impulse to be always doing or learning something which never left him through life. Although under no pressing need to work, and sent into the country for relaxation, he devoted himself

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[1] Minutes of Proceedings Inst. C.E., vol. lxxxi. p. 334.
[2] *Ibid*, vol. xxii. p. 618. [3] *Ibid*, vol. xix. p. 176.

steadily to an investigation in a branch of mechanics in which he had received no special instruction, and laboriously gathered information, for a remunerative application of which no opportunity was then visible. With characteristic modesty he made no mention of this work to Mr. Stephenson and was the more pleased, therefore, when in 1844 he received instructions from that gentleman to proceed to Dublin for the purpose of making a series of independent experiments on the working of the atmospheric system of traction, then in operation on the short line of the Kingstown and Dalkey Railway. The results of these experiments, made in conjunction with Mr. W. P. Marshall, formed an appendix to a Report on this subject by Robert Stephenson to the Directors of the Chester and Holyhead Railway Company and were embodied in a Paper entitled "The peculiar features of the Atmospheric Railway System,"[1] presented to the Institution by Mr. Berkley in the following year. He also made some further investigations on the London and Croydon Railway, which was then being worked atmospherically under the direction of Messrs. Samuda Brothers.

Another matter in which Mr. Berkley rendered considerable assistance was the Battle of the Gauges in 1846.[2] Three years previously he had carried out for Robert Stephenson an alteration of gauge in the Cambridge line of the Eastern Counties Railway from 5 feet 0½ inch to the ordinary width of 4 feet 8½ inches. Later, in 1873, when the gauge question was again before the Institution, Mr. Berkley communicated much valuable information, the outcome of thirty years' special experience.[3] In conjunction with the late Mr. George Henry Phipps,[4] he made for Robert Stephenson in 1850 a series of experiments and examinations of the deep wells of Liverpool with reference to the water-supply of that city. He was always on terms of intimate friendship with Mr. Stephenson, who placed the greatest confidence in his ability and at death left him a bequest as a mark of regard and esteem.

In the year 1849 Mr. Berkley commenced to practise on his own account. He designed and constructed Fenchurch Street Station and widened the line of the Blackwall Railway, for which Company he acted as engineer to the date of his death. He also constructed the Hampstead Junction Railway, the Hammersmith branch of the

[1] Minutes of Proceedings Inst. C.E., vol. iv. p. 251.
[2] *Vide* Report of Royal Commission in Library Inst. C.E.
[3] Minutes of Proceedings Inst. C.E., vol. xxxv. p. 385.
[4] *Ibid*, vol. xcvi. p. 330.

South Western, the Stratford and Loughton of the Great Eastern, the Wimbledon and Croydon, the East Suffolk system of the Great Eastern, the Wells and Fakenham and other lines. From the beginning of 1851 he acted as the representative of Robert Stephenson as consulting engineer to the Great Indian Peninsula Railway Company, to which post he was appointed on that gentleman's death in 1859. In 1867 and 1886 he visited India and, besides reporting on railways, was engaged in other engineering work in various parts of that empire.

In 1874 Mr. Berkley became one of the consulting engineers to the Colonies, for railways in Natal and viaducts in Cape Colony. In 1885 he was appointed consulting engineer to the Indian Midland Railway Company, and in 1887, in conjunction with his son, to a similar post on the Argentine North Eastern Railway. He was for many years one of the Board of Managers of the Royal Institution and a member of the Athenæum Club. In May 1893, he was created a Knight Commander of the Order of St. Michael and St. George. Unfortunately his health, which had not been good for some time, now began to show signs of serious failure. He had always an idea that his heart was not strong, and in this he was probably not far wrong, for he passed away suddenly during a fit of asthmatic coughing on the 20th of December, 1893.

An able engineer, Sir George Berkley owed much of his success to a capacity for taking advantage to the utmost of the many opportunities of advancement which fell in his way, and to the virtues of dogged perseverance and tenacity of purpose. Another marked feature of his character was incorruptible honesty in act and speech and a disinclination to acquiesce in anything of the nature of false suggestion or suppression of truth, even when such a course might have been followed by pecuniary benefit. His own estimate of his scientific and professional acquirements was always singularly modest, and even the importance of the position to which he ultimately attained did not serve to entirely remove this habit of self-depreciation.

Sir George Berkley's connection with the Institution began on the 1st of April, 1845, when he was elected an Associate. On the 17th of January, 1854, he was transferred to the class of Member; on the 17th of December, 1872, he was elected to a seat on the Council; and he served the office of President during the session 1891–92. In addition to the Paper on the Atmospheric Railway System, of which mention has already been made, he contributed to the Proceedings in 1870 a treatise entitled "On the Strength of Iron and Steel and on the Design of parts of Structures which con-

sist of those materials."[1] In his Presidential Address,[2] delivered on the 10th of November, 1891, he endeavoured to trace briefly the advance of engineering work in relation to social progress.

WILLIAM DEMPSEY was born in London on the 3rd of May, 1817, and was educated privately. He and his elder brother, George,[3] were early members of the Mechanics Institute founded by Dr. Birkbeck, and eagerly took advantage of the opportunity of self-education thus afforded. In 1840 he obtained an appointment as a draughtsman in the firm of Messrs. Fox, Henderson & Co. of Birmingham, with whom he remained until 1845.

In the latter year—at a time when railway enterprises were springing up all over the kingdom—William Dempsey joined his brother in London, where they worked night and day for some time in preparing plans for several projected lines. Unfortunately, however, the projects upon which they were occupied fell through, and William was glad to obtain an engagement in the works of Messrs. Bramah, Cochrane and Deeley at Tipton. In 1849 he was employed under Robert Stephenson[4] on the works of the Britannia Bridge. He then returned to Messrs. Fox, Henderson & Co., for whom he was engaged in the construction of the great Exhibition buildings of 1851, and subsequently in their reconstruction at Sydenham as the Crystal Palace.

Some two years later Mr. Dempsey started business on his own account and in 1857 was appointed Consulting Engineer in England to the Railway Department of the Government of South Australia. In that capacity he designed and superintended the manufacture of several iron bridges and roofs, among which may be mentioned the Murray River bridge, the Hamley bridge over the River Light, the Port Adelaide swing-bridge and an iron roof for the goods-shed at Adelaide. He also prepared the requisite drawings and specifications, supervised the manufacture of and tested many thousand tons of permanent-way materials of all kinds. In 1863 he was also appointed Consulting Engineer to the Scottish Australian Mining Company in all matters connected with the providing in England of the machinery and plant required for its extensive coal mines in the Colony. In addition to these offices

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[1] Minutes of Proceedings Inst. C.E., vol. xxx. p. 215.
[2] *Ibid*, vol. cvii. p. 1. [3] *Ibid*, vol. i. (1838) p. 38.
[4] *Ibid*, vol. xix. p. 176.

he filled for a time that of Consulting Engineer to the Government of Queensland.

For some time before his death, which took place at Kenilworth on the 18th of October, 1893, Mr. Dempsey had lived in retirement. The predominant features of his character were great capacity for work and inflexible uprightness. In his business relations he was just and generous to a degree, but his natural modesty perhaps made him appear diffident and retiring. Socially he was a most amiable and cheerful companion. Mr. Dempsey was elected an Associate of the Institution on the 6th of March, 1866, and was transferred to the class of Member on the 15th of March, 1870.

JOHN BAILEY DENTON was born on the 26th of November, 1814, and was the second son of the late Mr. Samuel Denton, a well-known solicitor of Gray's Inn during the first half of the present century. After a brief period of schooling, the youth, who was destined by his father to become a surveyor, was articled in 1830, when just fifteen years old, to the late Mr. Jackson, agent to the then Lord Dacre at Barkway, near Royston, Herts. There he learnt the practical management of estates, together with field-surveying, and under Mr. Jackson's directions undertook the inclosure of many of the common lands in numerous parishes in the midlands and home counties, and became noted for the accuracy of his work and the neatness of his plotted surveys. Surveying, however, afforded hardly wide enough scope for Mr. Denton's active mind and he determined to become a civil engineer. In 1842 he turned his attention to the railway movement then in full swing, and with the late Mr. Brassey,[1] and under the late Mr. Locke,[2] M.P., and other eminent engineers, was associated with the construction of the Great Northern, the London and South Western, the Midland, the Oxford and Cambridge, and the Hitchin and Royston Railways.

About this time, too, Mr. Bailey Denton, who had a natural inclination towards agriculture, became one of the earliest and most active promoters of the application of collective capital to the improvement of landed property, and was elected one of the directors of the first drainage company, which was originated with that object, viz., the Yorkshire Land Drainage Company. But as

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[1] Minutes of Proceedings Inst. C.E., vol. xxxiii. p. 246.
[2] *Ibid*, vol. xx. p. 141.

this company, which was started in 1843, failed to establish itself, owing to the difficulties raised by mortgagees and other claimants having interests in entailed estates, he, in conjunction with the late Duke of Richmond and the late Mr. Philip Pusey, succeeded in obtaining an Act in the year 1845, recognising the principle of priority for improvement charges over all other mortgages and incumbrances. In the meantime the government of the late Sir Robert Peel had undertaken to deal with the matter on an enlarged basis, and in order to meet the depreciating effects of the Repeal of the Corn Law, the "Public Money Drainage Act" was passed, under which £4,000,000 of public money was advanced to the owners of settled estates for drainage purposes, to be repaid in twenty-two years on very easy terms. It was to supply the place of the "Public Money Drainage Act," the fund available under which was soon exhausted, that a private Act of Parliament was obtained, mainly by the efforts of Mr. Bailey Denton, in the year 1849, called the "General Land Drainage and Improvement Company's Act," under which the owners of settled estates acquired the power of charging their estates with the cost of agricultural improvements of every kind for a period extended to thirty-one years. He was connected with this company from the time of its establishment in 1850 to the end of 1892, when age and failing health obliged him to retire, after having seen a sum of over three millions of money expended through its agency, many of the largest and most important estates in England and Wales having benefited by this outlay. During this long period he was frequently applied to for information as to the working of the Improvement of Land Acts in this country with a view to their adoption in other countries, notably in France, Austria, and Italy; and he on many occasions advised as to the reclamation of large tracts of land abroad. The pamphlets and works written by him on the various subjects relating to the improvement of landed estates are too numerous to mention. Most of them, however, are to be found in the Library of the Institution, but amongst them, as worthy of special notice, are, "The Farm Homesteads of England," "The Theory of Under-Drainage," "The Agricultural Labourer," "Farm Labourers' Cottages," and "Technical Education in Village Schools," all of which attracted considerable attention at the time of their publication.

Soon after 1860 Mr. Denton came prominently forward as an earnest advocate of the storage of surplus water throughout the country for the benefit of the community at large, his frequent appearances as a witness before Committees of both houses of

Parliament, and before the Duke of Richmond's Royal Commission, showing the great amount of labour which he had devoted to mastering the details of each watershed throughout England; and he lived to see his views, that there was plenty of water for the supply of London in the basin of the Thames, provided the periodical excess waters were stored in suitable localities, supported by the recent Royal Commission appointed to enquire into the water-supply of the Metropolis.

After the water question came the difficult sewage disposal problem, and in 1871, Mr. Bailey Denton, who had for some time been carefully studying, on his own responsibility, Dr. Frankland's theory of sewage purification by means of intermittent downward filtration through carefully prepared natural soil, as expressed in the Report of the Rivers Pollution Commissioners, was requested by the Lord Justices of Appeal, in consequence of the continued pollution of the River Taff, to put this theory into practice at Merthyr Tydfil. This he successfully did by the selection and preparation of 20 acres of free soil at Troedyrhiew near that town. After the completion of the work—the first of its kind in England—the proceedings in Chancery were soon withdrawn owing to the remarkably pure effluent obtained, and for many years the sewage of over 25,000 people has been utilized and effectually purified on this limited area of land. Emboldened by his success at Merthyr, Mr. Denton lectured and wrote upon the purification of sewage by means of filtration through land on every possible occasion, with the result that it was not long before over 100 local authorities had sought his aid upon the sewage question, and examples of sewage farms, as laid out by his firm, are now to be met with in nearly every county. The towns of Barnsley, Kendal, Forfar, Dewsbury, Abington, and Great Malvern were amongst the first to follow the example of Merthyr, whilst amongst the latest may be mentioned West Derby and Northampton.

With regard to the vexed question of the sewage-disposal of the Metropolis, Mr. Bailey Denton—whose design for the sewerage of London had, in 1849, been placed second on the list, out of 150 plans submitted to the judges, the first honours being awarded to the late Mr. McClean,[1] Past President—gave very lengthy evidence before Lord Bramwell's Royal Commission in 1885, and, with Colonel Jones, V.C., was the Author of the Canvey Island scheme, which was approved by the commissioners in opposition to the discharge of the liquid sewage into the Thames at Barking and

[1] Minutes of Proceedings Inst. C.E., vol. xxxviii. p. 287.

Crossness. His proposal was to purchase the Island and to dispose of the sewage there by filtration, and instead of barging the sludge out to sea to raise the surface of the land with it up to the level of the surrounding sea-wall. Mr. Denton maintained that by this means a valuable property would be created for the benefit of future ratepayers, the waste of valuable manure would be prevented, and that when all the surface of the island had been raised, the same method of procedure could be carried out on the low lands of Essex, near the mouth of the river. His principal works on the sewage question are: "Sanitary Engineering," a series of lectures given before the School of Military Engineering at Chatham in 1876, and "Fourteen Years' Experience of Intermittent Filtration," both of which are to be found in the Library of the Institution. He also wrote a Prize Essay for the Royal Agricultural Society entitled "On Land Improvements—Drainage, Farm Buildings and Cottages; by loans from Government or Public Companies."

Mr. Denton finally retired into private life in 1892, leaving his son and partners to carry on the professional work in which he had been engaged for upwards of sixty years. He died on the 19th of November, 1893, at Stevenage, Herts, where he had been a resident for more than fifty years and a county magistrate for nearly thirty.

Mr. Denton was elected an Associate of the Institution on the 5th of April, 1842, and was transferred to the class of Member on the 24th of January, 1860. Shortly after his election as an Associate he presented a Paper "On the Construction of Model Maps as a better mode than Sectioplanography for delineating the Drainage and Agricultural Improvements of a Country, or projected lines of Railways, Canals, etc." [1]; and in 1861 he read a second, entitled "On the Discharge from Under-Drainage and its effect on the Arterial Channels and Outfalls of the Country," [2] for which he was awarded a Telford Medal.

CHARLES LAMBERT DEPREE, eldest son of Mr. Charles Templer Depree, Solicitor, was born in London on the 19th of February, 1845. After commencing his engineering education in the Applied Sciences Department at King's College, where he

[1] Minutes of Proceedings Inst. C.E., vol. ii. (1842), p. 155.
[2] *Ibid*, vol. xxi. p. 48.

obtained several prizes, he was articled in 1864 for three years to Mr. John Arthur Wright, under whom he was engaged on the construction of the Kidwelly Branch of the Carmarthen and Cardigan Railway, and subsequently on that of the Doncaster and Gainsborough line. He was next entrusted, in 1868, by Mr. W. Banks of Paris with the charge of a French process for coppering iron armour-plated ships.

In the following year Mr. Depree determined to try his chance in the colonies and, armed with several testimonials and letters of introduction, proceeded to Queensland. He arrived there, however, at a time when the colony was still suffering from the effects of a financial crisis and public works were at a standstill. After passing the Government examination and qualifying as a Crown Lands Surveyor, Mr. Depree opened an office in Brisbane as a Civil Engineer and Surveyor. About that time he introduced into the colony concrete as a building material, having first erected a house for himself to demonstrate its advantages. He at once obtained a contract for the extension of one of the State asylums, which was most satisfactorily carried out. In July, 1871, Mr. Depree entered the Government service, his first work in which was to design and take charge of the improvement of Ross Creek, Townsville. In the following year he reported upon some obstructions in the upper reaches of the Mary River and prepared a scheme for their removal, the chief feature of which was a system of locks. From that time until 1875 he acted first as Assistant Engineer, and subsequently as Resident Engineer, on the Brisbane extension of the Southern and Western Railway, under Mr. Henry C. Stanley and Mr. J. Thorneloe Smith. From 1875 to 1877 he was Resident Engineer in charge of the working-survey of the Warwick and Stanthorpe Railway to the border of New South Wales, a distance of 42 miles; and from 1878 to 1880 he occupied the position of District Engineer in charge of the construction of the Maryborough and Gympie Railway, about 60 miles in length.

For the next three or four years Mr. Depree was engaged on trial- and working-surveys of several lines in Southern Queensland, and in 1884 became Assistant Inspecting Surveyor of the Southern Division of the Railway Survey Branch. Two years later he was promoted to the post of Assistant Engineer and placed in charge of the Southern and Central Division of the Railway Survey Branch. In that capacity he was responsible, immediately under the Chief Engineer, Mr. Stanley, for the whole of the railway surveys in the amalgamated division. Un-

fortunately, however, anxiety and pressure of work caused his health to break down. In 1890 he obtained twelve months' leave and came to England, hoping to derive benefit from rest and change in a more bracing climate. He consulted several eminent physicians, who all told him that a long rest in a colder climate than that of Queensland was absolutely essential for the restoration of his health. Unfortunately this change had not the desired effect and, after a residence in this country of a little more than three years, he died at Southport on the 30th of August, 1893.

Mr. Depree was unostentatiously kind and sympathetic in all relationships of life and by his conscientious and honourable conduct gained universal respect, while his frank and good-natured disposition endeared him to a large circle of friends in all parts of the colony. He was elected an Associate Member of the Institution on the 31st of May, 1881, and was transferred to the class of Member on the 15th of November, 1889.

BRYAN DONKIN was the fifth son of Bryan Donkin, F.R.S., the inventor of the first practical machine for making paper in a continuous web, and one of the earliest members of the Institution. He was born in the neighbourhood of his father's engineering works at Bermondsey on the 29th of April, 1809, and was educated at Bromley, Kent, at various schools near London, and at Paris and Nantes. At an early age he entered his father's works and gradually rose till he became the senior partner in the firm. From the first he was interested in all matters connected with the profession and he soon became an active and valuable assistant to his father. He helped in the design and construction of paper-making-, printing-, pumping-, and other machinery, and was much occupied in valuation and arbitration cases. He well remembered the first triangular-bar lathes; three were made, one for the Bermondsey works, one for Maudslay's, and the third for another firm. Before planing-machines were known it was necessary to make three lathes at a time, in order to get the bars perfectly true.

It was at this early period that he was brought into contact with several eminent engineers, among whom may be mentioned Isambard Kingdom Brunel,[1] Sir Henry Bessemer, John Penn the elder,[2] John Hall, Joseph Maudslay, Joseph Bramah, and John

[1] Minutes of Proceedings Inst. C.E., vol. xix. p. 169.
[2] *Ibid*, vol. iii. p. 13.

Farey,[1] author of the well-known work on the steam-engine, whose nephew, the late B. W. Farey,[2] was afterwards a partner in the Bermondsey firm.

In 1829 Bryan Donkin was sent to France by his father, to superintend the erection of paper-milling and other machinery at Nantes, and while there acquired great proficiency in the language of the country. The French Revolution of 1830, which occurred during his stay abroad, left an indelible impression on his memory. A few years later he surveyed the River Ebro in Spain and also made surveys for several railways in England. He had a distinct recollection of the first railway constructed. On the death of his father he became a partner in the Bermondsey works, at first in conjunction with his brothers, John [3] and Thomas, and afterwards with his nephews, Bryan Donkin, Jun., and E. B. Donkin, and B. W. Farey. He was an active and zealous worker and at all times keenly interested in every matter connected with engineering. He took out several patents for paper-making machinery and steam-engines, either alone or in conjunction with Mr. Farey, and assisted in the manufacture at Bermondsey of both Babbage's and Schultz's calculating-machines. In 1841 he married Miss E. Day of Isleworth, by whom he had three children.

In 1858 the firm accepted an important contract from the Russian Government, to construct and set up in St. Petersburg a mill to supersede the existing hand-mill for the manufacture of State papers and Government bank-notes. The undertaking was on a very large scale. It was necessary not only to superintend the construction of all the buildings, but to erect works for pumping water from the Neva, a mile distant, with settling-ponds and sand-filters. The latter were the first established in St. Petersburg and proved very efficient. The carrying out of these works took Bryan Donkin several times to Russia, and on their successful termination in 1862 he received the personal thanks of the Czar through the Minister of Finance. He had the satisfaction of witnessing their completion in company with General Winberg, at that time head of the Russian State Paper Department.

Mr. Donkin retired from active work in 1881, but continued to take a great interest in all business affairs up to the time of his death, which occurred at Blackheath on the 4th of December, 1893. His tact, business capabilities and many amiable qualities endeared him to those with whom throughout his long life he was

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[1] Minutes of Proceedings Inst. C.E., vol. xi. p. 100.
[2] *Ibid*, vol. xciv. p. 298. [3] *Ibid*, vol. xiv. p. 130.

brought in contact, and he ably sustained the reputation of the firm founded by his father.

He was elected an Associate on the 10th of March, 1835, was transferred to the class of Member on the 18th of February, 1840, and at the time of his death was the father of the Institution.

PERCY RICKARD, son of Mr. W. Rickard of Derby, was born in that town on the 12th of March, 1859, and was educated at the Grammar School there. He gave early indication of a practical turn of mind and at his own desire was placed, when not quite fifteen years of age, as a pupil in the Locomotive Works of the Midland Railway in his native town, under Mr. S. W. Johnson. Not only did he go through the shops and drawing office with great credit, but he carefully prepared himself for the future by attending in his spare time the local classes held under the auspices of the Science and Art Department. On the expiration of his pupilage he was articled in November, 1877, to Mr. Edward Parry of Nottingham, who was then acting as one of the Resident Engineers on the construction of the Nottingham and Melton branch of the Midland Railway.

After spending two years and a half with Mr. Parry, Percy Rickard entered, in April, 1880, the service of the Lancashire and Yorkshire Railway Company, under Mr. William Hunt. He was first placed in the drawing office at Manchester and was entrusted with making surveys and contract drawings, the most important being those for an iron bridge carrying one of the main thoroughfares in that city over the widening of the Victoria Station. He subsequently made for Mr. Hunt a survey of Fleetwood Channel, marking the contour lines of the bottom of the channel for every fathom in depth and indicating the direction of the currents. This was extremely well done. In January, 1883, Mr. Rickard was appointed Divisional Engineer in charge of about 150 miles in the Yorkshire district. He was responsible for the maintenance of the permanent way, station buildings and signals on that section, and also had charge of small contracts and extensions. In July, 1885, however, on the abolition of districts and the appointment of a Permanent-Way Engineer in charge of the whole line, he left the service of the Lancashire and Yorkshire Railway Company.

Mr. Rickard was not likely, however, to remain long idle. In the following year he was appointed—by his old master, Mr. Edward Parry—Resident Engineer on the construction of the

Nottingham Suburban Railway, a line of 4 miles on which are no less than four tunnels and several heavy iron bridges. On the completion of that work, he became in 1888 Resident Engineer for Messrs. Parry and Story on No. 1 contract of the Dore and Chinley line of the Midland Railway, which, although only 21 miles in length, reduces the distance between Sheffield on the one hand and Manchester and Liverpool on the other by no less than 32 miles in comparison with the old route round by Ambergate. The section of which he had charge was 10½ miles in length and included the Totley Tunnel, connecting the valleys of the Sheaf and Derwent. This tunnel is over 3½ miles long, being, therefore, second only in length in this country to the Severn Tunnel. Mr. Rickard has described it fully in a Paper entitled "The Tunnels of the Dore and Chinley Railway," which was read and discussed at the Institution on the evenings of the 23rd and 30th of January, 1894.

The circumstances of Mr. Rickard's untimely death are extremely sad. It seems that the brook into which the navvies' huts at the tunnel had been drained had during the past warm summer run nearly dry, leaving it practically an open sewer. This brook flowed through his garden and under a corner of his residence, and his doctors were of opinion that its foul state was probably the cause of the attack of typhoid which, on the 31st of October, 1893, unfortunately cut short a career of much promise.

Mr. Rickard was elected an Associate of the Institution on the 13th of January, 1885, and was transferred to the class of Member on the 27th of January, 1891.

ARCHIBALD SUTTER was born in Edinburgh on the 26th of January, 1829. At the age of sixteen he was apprenticed to Mr. Thomas Carfrae of that city for five years, during which period he superintended the erection of embankments of the Rivers Almond and Gogar. In 1854, some four years after the expiration of his pupilage, he started business on his own account in Edinburgh.

Amongst the works carried out from Mr. Sutter's designs and under his superintendence, the following may be mentioned:—the diversion of the Rivers Devon and Medwyn; the erection of bridges over them in 1856, 1864, and 1870, and the embankment of the former river at Dollar; drainage and waterworks for the town of Clackmannon in 1865–66; branch mineral railways at

Tillycoultry and Midcalder in 1867; drainage works at Alloa between Carsebridge and the River Forth in 1868; the Hopetoun waterworks, and the embankment of the Forth between Alloa and Kennet in 1869–70; the Kirkliston reservoir in 1873; water-supplies for the villages of Penstone and Airth, and a jetty at Alloa in the following year; sea-walls at Pitmilly in 1877; and since 1878 waterworks at Castle Douglas and at Winton, the repair of the pier-head at Dunbar Harbour, Newliston Bridge, Prestonpans drainage works, Dunbar waterworks extension, and surveys and soundings for the Forth Bridge.

In 1863 Mr. Sutter was appointed an Inspector to H.M. Enclosure Commissioners, an office he held for many years. He also acted as Parliamentary engineer for the new dock at Alloa and was engaged upon the surveys for the Edinburgh and District Waterworks. He had considerable experience too of railway work, notably in connection with the Edinburgh and Glasgow, the Edinburgh Suburban, and the Callander and Oban lines. Some few years before his death he opened an office in Westminster.

Mr. Sutter died in Edinburgh on the 30th of August, 1893, from heart disease complicated by other troubles. As an engineer he was conscientious and distinguished by the readiness with which he grasped the details of any scheme and by his quick perception of its salient points. He was ingenious also in devising expedients for overcoming difficulties which presented themselves from time to time. He took a prominent part in local and municipal affairs, was for some years a Town Councillor, and interested himself specially in social and sanitary movements. A staunch adherent of the Church of Scotland, he was for many years an office-bearer in the parish church of St. Andrew, Edinburgh. He was genial and courteous in manner, and not the least noticeable trait of his character was the kindly interest he exhibited in the studies and welfare of his assistants. He had occasion to visit the United States in 1881 and again in 1885, and the Australasian Colonies in 1883–84, and published his impressions in the form of books, entitled respectively "American Notes" and "Per Mare, per Terras." [1]

Mr. Sutter was elected an Associate of the Institution on the 5th of May, 1874, was placed in the class of Associate Member on its creation in 1878, and was transferred to that of Member on the 24th of April, 1883.

[1] Library Inst. C.E.

FREDERICK HENRY TREVITHICK, born on the 6th of November, 1843, was one of the well-known family of engineers of that name. His father, Mr. F. H. Trevithick, was Locomotive Superintendent of the Grand Trunk Railway of Canada; his uncle, Mr. Francis Trevithick, held a similar position at Crewe; and his grandfather was Richard Trevithick,[1] the introducer of the high-pressure steam-engine and the first to design and successfully work a locomotive on any line of railway.

When eighteen years of age, the subject of this notice commenced his professional education at the engineering works at Hayle, Cornwall, founded by his great-grandfather, Mr. John Harvey. On the expiration of his pupilage in 1862, he entered the drawing-office of the Locomotive Department of the London and North Western Railway at Crewe. Two years later, although only twenty-two, he was placed in entire charge of the railway between Frankfort and Homburg. That post, however, he resigned in 1866, to accept the more important appointment of Superintendent and Manager of the Danish Railways, which position he held until the lines were taken over by the State two years later.

After travelling for a short time in the United States, Mr. Trevithick was engaged from 1869 to 1871 in the Locomotive Department of the Central Pacific Railway at Sacramento, California. In the latter year he returned to England and was occupied for a time on independent construction work at Cardiff. In 1874 he was appointed Engineer and Manager of the Isle of Man Railway, but after holding that post for about two years he went to India as Locomotive Superintendent of the Madras Railway. With the exception of twelve months' leave in England in 1884 and eighteen months in 1889–90, Mr. Trevithick carried out the arduous duties of this appointment until April, 1891, when continuous ill-health compelled him to resign. He had in the previous year acted during the absence of the late Mr. F. B. Hanna[2] as Agent and General Manager. Unfortunately change and rest did not produce any permanent benefit and he died at Exeter from asthma and pulmonary consumption on the 19th of September, 1893, at the comparatively early age of forty-nine.

Mr. Trevithick was an exceptionally clever draughtsman and was never so happy as when solving some difficult or intricate

[1] Minutes of Proceedings Inst. C.E., vol. xxxvi. p. 99, and "Life of Trevithick," E. F. & N. Spon, London, 1872.

[2] *Ibid*, vol. cix. p. 403.

problem in mechanical design. Exceedingly fair and upright in his dealings, he was much liked by all his men, and though his constant ill-health and retiring disposition prevented his mixing so much with those around him as he might under other circumstances have done, he was untiring in his efforts for the general welfare of his subordinates. He took a keen interest in volunteering and to his efforts is due the formation and present state of efficiency of the corps of Madras Railway Volunteers, of which he was Commandant from its inauguration to the time of his retirement from the service.

Mr. Trevithick was also a Fellow, and examiner in engineering, of the University of Madras. Possessed of much general information, he was most interesting in society when his natural reserve allowed him to do himself justice. He suffered acutely from chronic asthma, an affliction he bore with an uncomplaining courage which was the wonder and admiration of all who knew the pain and weariness it entailed. On retiring from the Madras Railway Company he received a bonus in consideration of the exceptional services he had rendered in connection with the re-design and construction of the greater part of the wagon- and carriage-stock, nearly the whole of which was rebuilt at the Perambore works under his regime; and was presented with a farewell address and a handsome silver bowl by the employees of the Locomotive Department, and with a silver casket containing an address by the Madras Railway Volunteers. Speaking at the last half-yearly general meeting in London of the Madras Railway Company, the Chairman referred to Mr. Trevithick as "our late valuable and accomplished locomotive superintendent, who served us in that capacity very ably and faithfully for a period of sixteen years."

Mr. Trevithick married the eldest daughter of Captain Alexander Goldie, R.N., of Douglas, Isle of Man, who survives him.

He was elected a Member of the Institution on the 19th of May, 1885.

HARRY MACDONALD BECHER, son of General S. H. Becher of the Bengal Staff-Corps, was born at Simla on the 29th of January, 1855. He was sent to England at an early age and was educated at various private schools. At thirteen he was placed at school in Dresden for three years and then became a student at the Royal Mining Academy of Freiberg in Saxony. Returning to England in 1873, he went through the regular

course at the Royal School of Mines, of which he gained the diploma of Associate two years later. From October, 1875, to October, 1878, he was engaged at Patani in the Malay Peninsula as Engineer and Manager to the Galina Mining Company of Singapore, and had control during that time of over 500 men. He was then for four years in the employment of the Borneo Company at their Antimony Mines at Busao, where he had charge of more than 1,000 men and was engaged in making roads and tramways and in mining and smelting operations generally.

In 1882 Mr. Becher visited China and Japan, and, subsequently, surveyed and reported upon the coal deposits of Vladivostock Amoor in Eastern Siberia. On the completion of that work he spent some little time in England and, after inspecting various mines in the south of Spain, returned in November, 1883, to China to examine and report upon the mineral resources of Korea. He subsequently spent nearly three years in the interior of China inspecting mines, and in 1887 started, at Chantung, the first Chinese gold mine and quartz-mill. He then reported on gold mines in the north-east of Siam and early in the following year was engaged at the Punjom Pahang gold mines.

Mr. Becher's promising career was, however, unfortunately brought to an untimely end. While engaged on an expedition for the Royal Geographical Society to explore a mountain known as the Gunong Tahan and supposed to be the highest in the Malay Peninsula, he was drowned in the River Tahan on the 16th of September, 1893.

Mr. Becher was a Fellow of the Royal Geographical Society and of the Geological Society, and a Member of the American Institute of Mining Engineers. In June, 1892, he read, at a meeting of the Institution of Mining and Metallurgy, a Paper on "Mining in the Malay Peninsula." His frank, manly disposition, gained him many friends, among whom he was extremely popular. He was elected an Associate Member of the Institution on the 29th of May, 1883.

CHARLES RICHARD ERNEST CROOK, son of Mr. John Crook of Northam, near Southampton, was born on the 17th of September, 1863, and was educated at the School of Engineering attached to the Hartley Institution at Southampton, where he obtained a special prize for general work. He was then engaged for some two years in superintending various contracts carried out by his father's firm, among which may be mentioned a new quay and coal-

tramway for the Southampton Gas-light Company, and a new road bridge at Brockenhurst for the county authorities.

In May, 1882, Mr. Crook was appointed an assistant on the staff of the District Engineer at Clapham Junction of the London and South Western Railway. He remained in the service of that company for seven years, during which time he was engaged on various widening works both on the main and the Windsor lines, on new curves at Staines, Weybridge and Feltham, and on the re-building of Guildford and Woking stations. He then proceeded in August, 1889, to South America, to take up the post of a draughtsman in the engineer's office of the Central Argentine Railway Company. Unfortunately, however, heart disease developed and three years later Mr. Crook was forced to resign his appointment and return to England. A few days after his arrival he died at his father's home at Northam, Southampton, on the 18th of July, 1892, leaving a widow and one son. He was much attached to his work and the engineers under whom he acted had a high opinion of his character and ability.

Mr. Crook was elected an Associate Member of the Institution on the 4th of December, 1888.

LEWIS FREDERICK EVELEGH, youngest son of Captain George C. Evelegh, late R.A., of Newport, I.W., was born on the 17th of May, 1858. He commenced his engineering career in May, 1876, when he was articled to Mr. Henry E. Wallis, who was then practising in Westminster. After serving a pupilage of three years, he was sent in April, 1879, by Messrs. Wallis and Edge to Germany, where he was engaged until June, 1881, in laying the Brunswick Tramway and in superintending the manufacture of the rails for it at the Union Works, Dortmund. He was then employed during the next four years by the same firm in preparing plans and specifications for the Aston and Birmingham Tramway, in making surveys and Parliamentary plans for the Edgbaston and Harborne Tramway, and in preparing drawings for the Cardiff Station roof.

Towards the end of 1885 Mr. Evelegh proceeded to South America, where he was engaged during the next seven years, first on extensions of the Buenos Aires and Rosario Railway, and subsequently as Engineer to Messrs. Grant, Powell and Clarke on a contract for an extension of the Great Southern Railway of Buenos Aires. In the spring of 1893 he returned to England, when unfortunately heart disease developed. Anxious to adopt

the best means of recovery he became a paying patient at Guy's Hospital, where he died on the 17th of August, 1893.

Mr. Evelegh was elected an Associate Member of the Institution on the 13th of January, 1885.

HODGSON MONTEITH LAYARD JONES, eldest son of the late Mr. John Hodgson Jones,[1] was born on the 31st of October, 1843. At the early age of fourteen he was articled to his father for six years, after which he was placed in charge in 1864 of the extension of the gasworks at Malta. From June, 1865, to February, 1866, he was engaged at Glasgow and at Paisley inspecting work for shipment for the Continental Gas and Water Company and for the Malta and Mediterranean Gas Company. In the following April he undertook for the latter company the control of the erection of the Trapani Gasworks in Sicily, on the completion of which he constructed similar works at Marsala.

Returning to England in June, 1867, he was engaged for a time inspeeting for the Continental Union Gas Company, but in September, 1869, he commenced to practise on his own account in London. Finding competition very keen he wisely accepted in 1874 an appointment with the Oporto Gas Company, which he held for ten years. On the death of his brother, William,[2] in 1884 he became engineer and manager of the Popolo station at Rome of the Anglo-Romano Gas Company. He performed with credit the duties of that post until his death, which took place suddenly on the 27th of August, 1893, the cause being diabetes and symptoms of paralysis of the heart.

Mr. Jones' exceptionally kind and frank nature endeared him to a large number of friends. He was elected an Associate of the Institution on the 2nd of May, 1871, and was subsequently placed in the class of Associate Member.

RICHARD CAIL was born at Gateshead-on-Tyne on the 11th of May, 1812. After being educated at Mr. Bruce's school at Newcastle, he was—in accordance with the wish of his father, who thought that all youths should learn a trade—bound apprentice for seven years to Mr. Joseph Grey, builder, of that town. His master was a strict man of business and every day he

[1] Minutes of Proceedings Inst. C.E., vol. cxii. p. 353.
[2] *Ibid*, vol. lxxxii. p. 385.

was absent from work had to be made up before his indentures were delivered to him. On the expiration of his apprenticeship he commenced work on his own account in 1832 as a builder. He soon began to undertake considerable contracts and, being exempt as a Freeman of the town from the payment of all dues on cargoes, was able to make large profits on the timber, slates and Aberdeen flags which he imported in rapidly increasing quantities.

As he became better known, Mr. Cail obtained several important railway contracts, among which may be mentioned the construction in 1843 of the Sherburn and Shincliffe viaducts on the Newcastle and Darlington Junction Railway; the extension to Tynemouth of the Newcastle and North Shields Railway in 1845–6; the construction of the Richmond and Boroughbridge branches of the Great North of England Railway and the Warkworth branch (now the Amble branch) of the Newcastle and Berwick Railway in 1846–8; and the extension to Sleekburn of the Blyth and Tyne Railway in 1849–50. All these lines were eventually merged in the North Eastern Railway, for which company he constructed in 1853–6 the line from Leamside to Durham and Bishop Auckland, and subsequently the Dearness Valley Railway, the Rosedale branch, the Nidd Valley branch, and the connecting line between Nunthorpe Junction and Ingleby (now Battersby) Junction. He also constructed for the Whittle Dean Water Company (now the Newcastle and Gateshead Water Company) the Great Northern reservoir at Whittle Dean and the aqueduct tunnel through the mountain limestone ridge at Ryal.

In 1862 Mr. Cail gave up work as a contractor and became a member of the Walker Chemical Company, of which he acted as managing partner for several years. Subsequently he became interested in the sheet-iron works at Redheugh and was also chairman of the Redheugh Bridge Company. In 1876 he reported to the Tyne Improvement Commissioners on the Tyne piers, of which he proposed some alteration in the future mode of construction.[1] He subsequently became a prominent member of that Commission. Mr. Cail was for many years a member of the Town Council of Newcastle and twice served the office of mayor. He took a keen and practical interest in all municipal matters, especially as to the question of the local water-supply. For nearly twenty years he was on the Commission of the Peace for the city and county of Newcastle and was also a magistrate for the county of Durham.

[1] Library Inst. C.E., Tracts folio, vol. 24.

In spite of impaired sight and increasing feebleness Mr. Cail led an active life to the last. On the 18th of October, 1893, he attended a meeting of the Newcastle Town Council, at which he presented a report on his scheme for supplying the city with water from Lake Ullswater. He became ill during the meeting, but was able to drive home and go to bed. In a few hours, however, he died painlessly, from failure of the heart's action, in the eighty-second year of his age. Mr. Cail's career is an instance of what may be achieved by energy and perseverance. He was respected for his integrity and esteemed for his public spirit.

Mr. Cail was elected an Associate of the Institution on the 7th of December, 1875.

FREDERICK RANSOME,[1] born on the 18th of June, 1818, was one who, by patient research, enriched the world's knowledge of the chemical treatment of minerals. Early in life the idea occurred to him that stone, like iron, might be melted and run into moulds, and that the myriads of little grains of sand by the sea-shore might be cemented indissolubly together. So long ago as 1848 he presented to the Institution a Paper entitled "On the Manufacture of Artificial Stone with a Silica base."[2] After many years of experiment, during which he made several discoveries, some of them unexpectedly, he perfected his process of manufacture, which was based on one of the most beautiful of chemical reactions. Flints were first dissolved by means of caustic alkali under high pressure, so as to form soda silicate—a kind of water glass. This viscous and tenacious substance was then rapidly mixed with a proportion of very fine and sharp silicious sand in a vessel, so as to form a soft plastic mass which could be moulded into any shape desired. The soft stone was then placed in a bath of calcium chloride solution, which was made to penetrate every pore by means of hydraulic or atmospheric pressure. When this solution came into contact with the soda silicate a double decomposition took place, the silica combining with the calcium and forming a hard solid silicate of lime, and the soda uniting with the chlorine to form sodium chloride in a small quantity. Instead, then, of the particles of sand being covered with a thin film of the liquid silicate of soda, they were covered and united together with a film of solid silicate of lime, one of the

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[1] The substance of this notice is taken from "Engineering," vol. lv. p. 624.

[2] Minutes of Proceedings Inst. C.E., vol. vii. p. 57.

most indestructible substances known. The small quantity of soluble sodium chloride, one of the results of decomposition, was then washed out of the stone by a douche of clean water or by hydraulic pressure, and when dry the stone was ready for use. A cube of stone 4 inches square took 44 tons to crush it, although only ten days old, while sections 2¼ inches square bore respectively 870 lbs.[1] and 1,200 lbs.

The artificial material produced by Mr. Ransome was a sandstone, the silicious particles of which were bound together by a cement of silicate of lime. Its composition was precisely that of the best building stone known—such as Cragleith and some varieties of Yorkshire stone, which resist the most trying air and climate. When fractured it showed perfect homogeneity, so that it was admirably adapted for carving with chisels. It could be moulded into the most delicate forms while in a soft state, and could be surface-dressed or finished when hard, if necessary. Its plasticity during the first process of manufacture enabled it to be used with great economy in all elaborate mouldings and repeated ornamentation, and many of the public buildings in England, Bombay, Calcutta, and New Zealand may be regarded as memorials of Mr. Ransome's success, such as the new India Offices, St. Thomas's Hospital, the Brighton Aquarium, the Albert Bridge at Chelsea, and many churches; while for other purposes, as in the Metropolitan Railway stations and at the London Docks, the material was largely used.

The process was utilized for many other purposes, devised in succeeding years by Mr. Ransome, notably for emery wheels and for grindstones. He also constructed a filter, in which the water was passed through porous slabs. While engaged in such minor pursuits, the subject of this memoir continued to direct his studies to the manufacture of other materials, one of the results being the manufacture of cement from blast-furnace slag and lime. Into the details of the process it is scarcely necessary to enter, further than to state that he worked assiduously until he perfected it, overcoming all difficulties and producing a cement the tests of which[2] showed that it possessed remarkable properties, the strength within a few days of manufacture being stated to be higher than that of Portland cement after seven years. Being made from waste materials, it could be produced at half the cost of Portland cement. He next directed his energy to the mechanical details for burning the cement, and produced a novel type of revolving

[1] "Engineering," vol. iii. p. 671. [2] *Ibid*, vol. xxxix. p. 95.

kiln, the characteristic feature of the process[1] being that the material was burned in a state of powder, and that it emerged from the furnace in this condition, the object being to reduce the final grinding from a tedious and expensive process to a very simple and rapid operation.

Mr. Ransome spent the last few years of his life in retirement. He died on the 19th of April, 1893, at the age of seventy-five. His geniality and kindness of heart gained him many friends. In middle life he was a frequent attendant at the meetings of the Institution, of which he was elected an Associate on the 7th of March, 1848, and served as an Associate of Council in 1868–69.

COLONEL JOHN BARNES SPARKS, of the Bengal Staff Corps, was born at sea—off Sierra-Leone, West Africa—on the 19th July, 1841, his mother being on her way to join her husband, then Captain and afterwards Colonel George Mitchell Sparks, who was serving with his regiment, the 12th Foot, in China, and afterwards fell in India.

Educated in India, John Barnes Sparks early attained prominence in his studies at Mussouri, and then proceeded to the Thomason Civil Engineering College at Roorkee, where he highly distinguished himself in mathematics and engineering subjects, obtaining a gold medal and other distinctions. On the outbreak of the Mutiny, in 1857, he was enrolled among the volunteers, but did not take any part in the actual warfare. On the 5th of August, 1859, he received his first commission as Ensign in the 38th Foot Regiment; from which he was appointed to the Bengal Staff Corps, joining the Public Works Department in August, 1863. After being employed on surveying, road construction, and other work for some time, he was transferred to Gwalior, where, besides the construction of government works, he was engaged in the reparation of the Fortress which has been recently restored to Scindia. In 1868 he married Eliza Jane, widow of Frederick Kitchen Buist, of Cawnpore, and in 1870 visited England on furlough. In the following year he was promoted to the rank of Captain.

On returning to India towards the end of 1871, Captain Sparks was posted to Karachi as Assistant Engineer, 1st Grade, and was engaged on the construction of the Indus Valley State Railway, and in the management of the stores of that line. On promotion to Executive Engineer, 4th Grade, in 1872, he was moved to

[1] "Engineering," vol. xlii. p. 156.

Mooltan, still on the same railway. While engaged on this and the Kandahar Railways under Mr. J. R. Bell, he took part in the construction and opening of the Empress bridge over the Sutlej [1] at Adamwahan, which at that time was one of the most important engineering works in India. During his residence at Mooltan he established, among numerous undertakings of local interest, co-operative stores for the railway servants which, under his management, resulted in great benefit both to the employees and to the Railway Servants' Fund. He was promoted to the rank of Major on the 5th of August, 1879.

In the following year Major Sparks again visited England on furlough and shortly after his return was appointed to Bombay to the important post of port storekeeper to the State Railways Department. In this position, which he held for seven years, he showed great business capability and capacity for management. In August, 1885, he was promoted to Lieutenant-Colonel and four years later to full Colonel. After another brief visit home, and return to his post at Bombay, he spent two years in England, from May, 1889, at Sydenham. Here he speedily became known as an active helper in all charitable and philanthropic undertakings, and gained great popularity during his short residence.

In May, 1891, Colonel Sparks again returned to India, and was appointed Chief Engineer to the East Coast Railway of Madras. After a short time he undertook the management of the stores of that line, but had hardly entered upon these duties when his health broke down and under urgent medical advice he was forced to proceed home. He reached England in August, 1892, but he gradually declined and passed peacefully away on the 6th of March, 1893, at Addison Mansions, Kensington, at the age of fifty-two years.

During his thirty-two years of service in India in various positions, Colonel Sparks' industry and ability obtained for him a high reputation among his superior officers and companions. In private life he was held in the highest estimation by all who knew him. Especially in Mooltan and Bombay he took more than a mere interest in all works, whether of a social, general, or charitable character, and his aid and advice were always sought. He was also connected with the University of Bombay; and the Byculla schools of that city owe much of their success to his careful and able management as honorary secretary and treasurer.

Colonel Sparks was elected an Associate of the Institution on the 21st of May, 1867.

[1] Minutes of Proceedings Inst. C.E., vol. lxv. p. 242.

*** The following deaths have also been made known since the 14th of September, 1893:—

Honorary Member.

TYNDALL, JOHN, LL.D., F.R.S.; *born* 1820; *died* 4 December, 1893.

Members.

ALLISON, JOHN; *born* 26 February, 1838; *died* 13 February, 1894.
BURNS, JEROME; *died* 14 January, 1894, *aged* 68. (*Pneumonia.*)
BUTTERTON, WILLIAM; *died* 12 December, 1893, *aged* 76. (*Pneumonia.*)
ELLIOT, Sir GEORGE, *Bart.*, M.P.; *born* 18 March, 1815; *died* 23 December, 1893. (*Pneumonia.*)
GARDNER, JOHN; *born* 16 December, 1821; *died* 15 January, 1894.
GILROY, GEORGE; *died* 1 January, 1894, *aged* 70. (*Paralysis.*)
GRANT, JOHN DUNCAN; *born* 4 August, 1848; *died* 24 November, 1893. (*Heart disease.*)
HAWKSLEY, THOMAS, F.R.S.; *born* 1807; *died* 23 September, 1893.
HICK, JOHN; *born* 1814; *died* 2 February, 1894.
HOMER, CHARLES JAMES; *born* 17 August, 1837; *died* 4 November, 1893.
HYDE, MARK; *born* 13 March, 1823; *died* 10 May, 1893.
IKIN, JONATHAN DICKSON; *born* 23 July, 1826; *died* 24 September, 1893.
KNOWLES, JOHN; *born* 1 October, 1828; *died* 25 January, 1893. (*Congestion of the lungs.*)
LESLIE, ALEXANDER, F.R.S.E.; *born* 16 September, 1844; *died* 7 December, 1893.
LONGSDON, ALFRED; *born* 22 May, 1827; *died* 27 November, 1893.
PURCHAS, SAMUEL GUYON; *born* 19 November, 1823; *died* 12 February, 1894. (*Heart disease.*)
YOCKNEY, SAMUEL HANSARD; *died* 29 December, 1893, *aged* 80.
YOUNG, JOHN BROWN; *born* 30 April, 1842; *died* 6 June, 1893.

Associate Members.

BARBENSON, ROBERT THOMAS OLLIVIER; *died* 1893.
CARLTON, GEORGE BRODY; *born* 18 July, 1854; *died* 23 November, 1893. (*Typhoid fever.*)
DONAGHUE, JOHN; *born* 25 March, 1866; *died* 14 November, 1893. (*Congestion of the lungs.*)
ELLIOT, EDMUND COLVILLE; *born* 15 May, 1857; *died* 30 August, 1893.
ELLIOT, WILLIAM; *born* 24 November, 1856; *died* 25 October, 1893. (*Heart disease.*)
FAKIR CHAND, *Rai Sahib*; *born* 15 October, 1860; *died* 25 August, 1893. (*Pneumonia.*)
FOORD, ALFRED MONTAGUE; *born* 28 May, 1858; *died* 6 January, 1894.
MARTIN, WALTER RICHARDS; *born* 15 May, 1860; *died* 17 November, 1893.
NETTLEFOLD, HUGH; *born* 19 August, 1858; *died* 23 December, 1893. (*Pneumonia.*)
ROSS, HENRY FRANCIS; *died* 12 January, 1894, *aged* 74.
SMYTHIES, JOHN PALMER; *died* 3 January, 1894.
STEWART, JAMES, Jun.; *born* 11 August, 1863; *died* 26 January, 1893. (*Bite of a snake.*)

Associates.

LOUTTIT, SAMUEL HENRY; *born* 16 June, 1838; *died* 18 November, 1893. (*Congestion of the lungs.*)
LOVELACE, WILLIAM, Earl of, F.R.S.; *born* 20 February, 1805; *died* 29 December, 1893.
ROGERSON, JOHN; *died* 6 February, 1894, *aged* 65. (*Angina pectoris.*)

Information as to the professional career and personal characteristics of the above is solicited in aid of the preparation of Obituary Notices.—SEC. INST. C.E., 28 *February*, 1894.

Sect. III.

ABSTRACTS OF PAPERS IN FOREIGN TRANSACTIONS AND PERIODICALS.

The Base-line of the Prussian Ordnance Survey.

By Colonel Morsbach.

(Zeitschrift für Vermessungswesen, 1893, p. 1.)

The measurement of the new base-line for the Prussian Ordnance Survey at Bonn was effected with Bessel's apparatus in eleven working days (July 19 to 30). The old Bonn base-line, measured by General Baeyer with the same apparatus, was 2,134 metres long, and was situated on the high road from Bonn to Cologne. In the middle of the base-line the road is sharply curved, and it was therefore necessary to break the line, and the point at which this took place was marked under ground. The marks showing the positions of this point and of the two terminal points of the line were in 1888 apparently unaltered. For various reasons it was thought undesirable to use this old base-line for the new survey, and the site for the new line was selected in a field east of the road. The line is nearly parallel to the old base, at a distance of about 80 metres, and it extends to the north 380 metres beyond the old base. The new measurement enables the length of the side Birkhof-Michelsberg of the Rhenish-Hessian Triangulation to be determined absolutely, and it affords a comparison with results of the measurement carried out by the Geodetic Institute with Brunner's apparatus, and with those of the old measurement.

The new line was measured four times, twice from north to south and twice from south to north. With a view to obtain information regarding the metals employed (iron and zinc), two measurements were made with increasing temperatures (in the morning), and two with decreasing temperatures (in the evening). The base was divided into sixteen sections by means of subterranean fixed points, which were connected with the precision-levelling of the country. The measurement was carried out under the direction of the Author by six officers and eight officials, assisted by two non-commissioned officers, eleven sappers, and forty-two linesmen. The results of the measurement reduced to the Ordnance datum were as follows:—

	Metres.
First measurement	2,512·92747
Second measurement	2,512·92912
Third measurement	2,512·92638
Fourth measurement	2,512·92770
Mean . . .	2,512·92767

The mean instrumental error was determined to be 0·697 millimetre per kilometre, or 1·106 millimetre in the base-line when measured once. When measurements were made four times, the mean instrumental error was 1·006 millimetre per kilometre, or 0·553 millimetre in the base-line. The total cost of the base measurement was £665.

B. H. B.

The Value of the Metre in English Inches. By C. B. Comstock.

(American Journal of Science, vol. xlvi., 1893, p. 74.)

In 1885 the Author gave the value of the metre as 39·3699 inches, a result deduced from comparisons of a metre (R 1876), and Clarke yard A, both formerly belonging to the United States Lake Survey. The value of the metre derived from Dr. Benoit's comparisons is 39·3699 inches. That authority's note gives the value of the toise of Bessel in terms of the metre, of toise No. 10 in terms of the toise of Bessel from Peter's comparisons, and of toise No. 10 in terms of the yard from Clarke's comparisons. These equal values of the metre, obtained from independent sources, afford evidence that this value has a high degree of accuracy.

B. H. B.

The Testing-Station for Building-Materials at Zurich.

By L. Tetmajer.

(Schweizerische Bauzeitung, vol. xxii., 1893, p. 24.)

The origin of the present testing-station at Zurich was a small building erected in 1879 for the reception of some testing-machines which had previously been obtained for special purposes. Although this building was added to from time to time, it soon became insufficient for the growing wants of the public, and it was decided to erect a new one which, together with the apparatus it contains, is described and illustrated in this Paper. The building is of two storeys, with cellars which practically form another storey. The ground area covered by the building is 750 square metres (897 square yards). The machine-room is placed centrally, so that power may, if necessary, be transmitted to the various laboratories.

In addition to offices, store-rooms, &c., the following are the chief laboratories or workshops provided: Cement-testing room, wet-store room, smithy, dust-room, wet workshop, photographic

dark-room, physical laboratory, chemical laboratory, mechanical laboratory, machine-room, mechanics' shop, museum and lecture-room. The building is constructed of brick and stone, and for the floors in most cases Portland cement concrete has been used. In some of the rooms, however, special floors have been laid down suitable to the particular work to be carried on. The greater part of the rooms are heated by a low-pressure hot-water system, and are lighted by electricity, although gas is also used for special purposes. There are two 500-candle arc-lamps and sixty-six glow-lamps. There is a water-supply, with a pressure of 4·8 atmospheres, which is used as a source of power in some cases. The cost of the building alone was about £8,080, while the value of the whole establishment, including machinery, is £13,911.

Although the object of the station is simply to test building materials when necessary, and not to undertake research work, like the larger institutions in Berlin and Munich, yet the equipment of each department is complete. A full descriptive inventory is given by the Author, together with illustrations of the chief rooms.

W. F. R.

The Elastic Properties of Cements and Mortars.

By Professor E. Hartig.

(Der Civilingenieur, 1893, p. 435.)

This Paper contains a description of apparatus employed in the measurement of the elastic compressions and extensions of cements and mortars, and a discussion of the results of the experiments.

The apparatus is of the "roller and mirror" type; the reflection in the mirror of a fixed scale being read off by a telescope. In the simplest form of apparatus used, the test specimen has enlarged shoulders at the ends, in which have been embedded iron hooks, whose projecting ends can be attached to the clips of the testing machine. Attached by binding screws to one end of the specimen is a frame carrying the roller and mirror, to the other end a frame carrying a small conical cup. A long steel rod, or distance piece, rests with its pointed end in the bottom of the conical cup, while its other end passes through a hole in the frame carrying the roller and mirror. A short piece of thin steel ribbon has one end fixed to the distance piece, its other wrapped round the roller; so that the extension of the specimen produces a small rotation of the roller and mirror. A flat steel spring is arranged so that the apparatus is kept taut, the pressure between the cup and distance piece being regulated by a screw and nut.

In order to eliminate the effects of bending of the test specimen,

the extension of two opposite sides should be taken into account. In a second and more perfect form of apparatus the test specimen (tension) is moulded of uniform thickness, its width being increased at the ends for attachment in the jaws of the testing machine. The testing machine used is horizontal. A frame touching the specimen at three points is attached near each end by binding screws; the distance between the points of the two frames being the length whose extensions are measured. To one frame, at opposite sides of the specimen, the ends of two thin steel ribbons are fixed, and their other ends are passed round and fixed to small vertical spindles carried by the other frame. These spindles are continued a little above the top of the specimen, and have their ends increased in diameter. Two short steel ribbons have each one end wrapped round and fixed to the enlarged portions of the vertical spindles, and their other ends are attached to the ends of a small horizontal beam. From the middle of this beam another short steel ribbon is led round and fixed to the roller to which the mirror is attached. A spring suitably placed keeps the steel ribbons taut. Thus the rotation of the mirror is proportional to the sum of the extensions of two opposite sides of the specimen.

Results of Experiments.—With all the materials experimented on, it was found that the first application of load produced a considerable permanent set as well as elastic extension; the extensions due to repeated loadings less than the first were, however, perfectly elastic. The stress σ and the strain ϵ are not exactly proportional, so that the modulus of elasticity E is not constant. The relation between the stress and elastic strain may be expressed—

$$\sigma = a\,\epsilon + b\,\epsilon^2,$$

a and b being specific constants for each material. Then

$$E = \frac{\Delta\sigma}{\Delta\epsilon} = a + 2\,b\,\epsilon.$$

Plaster of Paris manufactured by Mr. L. Mundt, Berlin, gave the following results. Tensile resistance, 13·5 kilograms per square centimetre = 192 lbs. per square inch; compressive resistance, 118 kilograms per square centimetre = 1,678 lbs. per square inch; greatest elastic extension, $\frac{1}{341}$ of the original length; permanent set, $\frac{1}{595}$ of the original length;

$$\begin{aligned} E &= 10^5\,(0\cdot415 - 39\cdot0\,\epsilon) \\ \sigma &= 10^5\,(0\cdot415\,\epsilon - 18\cdot5\,\epsilon^2) \\ \epsilon &= 10^{-5}\,(2\cdot41\,\sigma + 0\cdot00259\,\sigma^2), \end{aligned}$$

E and σ being expressed in kilograms per square centimetre. The elastic properties of plaster of Paris may therefore be expressed by the following figures:—

—	σ		ε	E	
	Kilograms per Square Centimetre.	Lbs. per Square Inch.		Kilograms per Square Centimetre.	Lbs. per Square Inch.
Breaking point, tension .	13·5	192	0·00033	40,400	574,500
Zero stress	0	0	0	41,500	590,200
Breaking point, compression	−118	−1,678	−0·00281	49,500	704,100

The work done in extending the prismatic bar up to breaking point is 21·3 metre-kilograms per tonne weight of material, *i.e.*, 0·070 foot.

The work done in compression up to breaking point is 1,301 m.k.t. = 4·27 feet.

Keene's Marble Cement.—The stress-strain curve is approximately a straight line, and E = 98,850 kilograms per square centimetre. The maximum stresses and strains are—

—	σ		ε
	Kilograms per Square Centimetre.	Lbs. per Square Inch.	
Breaking point, tension	36·9	525	0·00037
„ „ compression . .	−411	−5,845	−0·00416

The work done in tension and compression respectively is 42·7 m.k.t. and 5,310 m.k.t.; *i.e.*, 0·140 foot and 17·4 feet respectively.

Portland Cement.—The length of the test specimen has a considerable influence on its tenacity. Nine different makes of Portland cement were made into mortars of the proportion 3 sand 1 cement; briquettes, 5 square centimetres sectional area and of the ordinary fiddle shape, and prismatic test-pieces 4 square centimetres section and of sufficient length to have the extensions of a 20-centimetre length measured, were made from each mortar and tested for tenacity. The strength of the long blocks was, on the average, 0·44 time that of the corresponding short briquette.

Besides experiments on elastic extensions and compressions, experiments to determine the other physical properties of the cement were made. The Portland cement made by Mr. E. Tillgner, Schimischow, may have its elastic properties represented as follows. For neat cement—

$$E = 10^5\,(1\cdot61 - 284\,\epsilon)$$
$$\sigma = 10^5\,(1\cdot61\,\epsilon - 142\,\epsilon^2)$$
$$\epsilon = 10^{-5}\,(0\cdot621\,\sigma + 3{,}400\,\sigma^2)$$

—	σ Kilograms per Square Centimetre.	σ Lbs. per Square Inch.	ϵ	E Kilograms per Square Centimetre.	E Lbs. per Square Inch.
Breaking point, tension .	45·2	643	0·00028	153,000	2,177,000
Zero stress	0	..	0	161,000	2,290,000
Breaking point, compression	−453	−6,443	−0·00232	227,000	3,229,000

The specific work done in breaking the bar is—

For tension . . . 32·5 m.k.t. = 0·107 foot.
For compression . 2,452 m.k.t. = 8·044 feet.

For cement mortar, 3 sand 1 cement—

$$E = 10^5\,(2\cdot655 + 2{,}774\,\epsilon)$$
$$\sigma = 10^5\,(2\cdot655\,\epsilon + 1{,}387\,\epsilon^2)$$
$$\epsilon = 10^{-5}\,(0\cdot377\,\sigma - 7{,}420\,\sigma^2)$$

—	σ Kilograms per Square Centimetre.	σ Lbs. per Square Inch.	ϵ	E Kilograms per Square Centimetre.	E Lbs. per Square Inch.
Breaking point, tension .	38·6	549	0·00013	302,700	4,305,000
Zero stress	0	0	0	265,500	3,777,000
Breaking point, compression	−127	−1,806	−0·00095	133,700	1,901,000

The specific work done in breaking the bar is—

For tension . . . 13·0 m.k.t. = 0·043 foot.
For compression . 425·7 m.k.t. = 1·396 foot.

From the above figures an average value of the coefficient of elasticity of cement mortar for practical use may be obtained. Taking the maximum working stresses, 1 kilogram per square centimetre for tension and 7 kilograms per square centimetre for compression, the corresponding values of E are 266,300 and 248,200, from which an average value

E = 257,000 kilograms per square centimetre,
= 3,656,000 lbs. per square inch,

may be taken.

This value is much higher than that usually assumed, viz., from 50,000 to 150,000 kilograms per square centimetre. Also since, from the foregoing investigation, an increase in the proportion of sand increases the modulus, the modulus of elasticity of concrete must be very much higher than commonly assumed. The Author hopes to give a Paper on this subject later on.

The Paper is accompanied by drawings of the apparatus, and a number of curves plotted from the experimental observations.

A. S.

Concrete Bridge over the Danube at Rechtenstein, Würtemberg.

By — Braun.

(Zeitschrift für Bauwesen, 1893, p. 439.)

At this place, situated on the Upper Danube, there is, amidstream, an island of Jurassic limestone rock, about 82 feet long and from 13 to 16½ feet broad, serving as the base for the centre pier of a road bridge, which, up to the year 1892, was formed by a timber-framed structure in two openings, each of 75 feet 6 inches span. The left one, of oak, had been erected in 1830, the right one had been renewed, in pine, by the State Railway Company in 1869.

In 1891 an investigation showed that the roadway of the left span had subsided 12 inches and that of the right span 6 inches, and that the structure would have to be replaced by another. In carrying this out it was essential that the utmost economy should be observed, as the cost would entirely fall on the small local community, and after full consideration it was decided to erect a concrete structure of two arches of similar span to those of the original structure.

The character of the rocky bottom of the stream was tested by bars of 1½ inch diameter, with enlarged pear-shape points of steel; these were driven with a sledge, and after every twenty blows, the amount of subsidence noted, thus showing, in an economical manner, whether piles could be driven for the pillars supporting the centering. For the left opening it was found that piles could be driven, but in the right opening that concrete pillars would have to be adopted. Each of the two spans is 75 feet 6 inches, the thickness of the pier is 8 feet 2 inches, the width of the arch is 11 feet 2 inches, and the depth of the latter at the crown and springing respectively 2 feet 1½ inch and 2 feet 11½ inches, and the versed sine 8 feet 2½ inches. The clear width between the parapets, which are formed by an iron railing, is 12 feet 1½ inch, the roadway being 7 feet 6½ inches broad, and the raised footways on each side 2 feet 3½ inches. The arches are compound curves, of 98 feet 6 inches and 65 feet 7 inches radius respectively, the latter near the springings. The roadway rises to the centre of the arch with

a gradient of 1 in $33\frac{1}{3}$. The strains upon the structure when fully laden are as follows:—

	Lbs. per Square Inch.
At the crown	222
,, joint of rupture	256
,, springing	185
,, foundation of left abutment	43
,, ,, ,, right ,,	14

The proportion of the cement, sand, &c., used in the various portions of the structure were as follows:—

—	Cement.	Sand.	Gravel.	Quantity of Cement per Cubic Yard of Concrete.
				Lbs.
Foundations	1	4	$8 + \frac{1}{4}$ broken stone	224
Arch	1	$2\frac{1}{2}$	$5 + \frac{1}{8}$,, ,,	460
Hinge lead-joint blocks	1	2	4	522
Above the arch	1 1	4 3	8 6	324
Footways, &c.	1	2	4	705

In this structure a system of hinged joints was adopted by leaving an open joint at the crown and near each springing and the introduction of a pad of rolled lead about their centre. The procedure adopted in making these joints is described, and the question discussed as to whether the resultant pressure on these, after striking the centres, was not considerably greater than that originally intended, viz., 85 lbs. per square inch. The amount of settlement of the crown of the arches, on striking the centres with sand boxes, was $1\frac{9}{16}$ inch in one case and $1\frac{3}{16}$ inch in the other.

The work was commenced in May and completed in October, or a period of five months. The whole cost of the bridge was £690.

The paper is illustrated by a general view and by a series of diagrams.

D. G.

On the Influence on the Flexure of Beams of the Superficial Position of the Load. By A. Flamant.

(Annales des Ponts et Chaussées, August 1893, p. 228.)

According to the ordinarily accepted theory of the distribution of stress in beams, the effect of a concentrated load is independent of its position on the vertical line along which it acts. The present investigation attempts a more accurate analysis of the

distribution of stress on the principles of the theory of elasticity, and appeals for confirmation of its results to the experiments of Professor Carus Wilson.[1]

The conditions of stress are first investigated of an elastic solid, bounded on its upper surface by a horizontal plane and extending indefinitely in all other directions, a vertical load being uniformly distributed along one line in the surface. If the axis of y be the line in the surface on which the load is applied, that of x lying also in the surface and that of z being vertical, the solution of the equations of equilibrium gives:

$$N_x = -\frac{2P}{\pi}\frac{x^2 z}{(x^2+z^2)^2},$$

$$N = -\frac{2P}{\pi}\frac{z^3}{(x^2+z^2)^2},$$

$$T = -\frac{2P}{\pi}\frac{x z^2}{(x^2+z^2)^2},$$

in which P is the load per unit of length of the axis of y and N_x, N_z and T have the usual meanings in the theory of elasticity.

It follows from the above equation that the resultant stress acting on any elementary plane surface at right angles to the plane $x\,z$ is directed towards the origin of co-ordinates (which is the point of application of P), and that the total horizontal stress across the plane $y\,z$, instead of being zero, as in the ordinary theory, is equal to $\frac{\pm P}{\pi}$.

In applying this theory to the case of a beam of definite thickness supported throughout, the above solution is complete if it is admitted that the reactions at every point of the continuous support are those which the infinite solid previously assumed would have supplied at the same points by its internal stresses.

When the beam, instead of being supported throughout, rests only on supports (assumed equally distant from the origin), an assumption must be made as to the distribution of the horizontal stress at various points on a vertical line. This distribution depends on the flexure, and the Author accepts the suggestion of Professor Sir G. G. Stokes that the horizontal stress may be assumed to be a linear function of the vertical ordinate z. This assumption, with the condition that the resultant of the horizontal stresses on the plane $y\,z$ is equal to $\frac{P}{\pi}$, and with the statical equation of moments suffices for the solution of the problem, and gives the result,

$$N_x = \frac{P}{h}\left[\left(\frac{4}{\pi} - \frac{3\,l}{h}\right) + 6\left(\frac{l}{h} - \frac{1}{\pi}\right)\frac{z}{h}\right],$$

where l is half the span, and h the depth of the beam.

[1] Philosophical Magazine, December 1891.

It follows from this that the neutral fibre (where $N_x = 0$) is not, as usually supposed, at the middle of the depth of the beam, but is at a distance r from the middle point, where

$$\frac{r}{h} = \frac{1}{6\left(\frac{\pi l}{h} - 1\right)}.$$

It is seen that when $\frac{l}{h}$ is considerable, $\frac{r}{h}$ is small; thus, if $\frac{l}{h} = \frac{11}{\pi}$ (or the depth one-seventh part of the span),

$$\frac{r}{h} = \frac{1}{60}.$$

The horizontal and vertical shearing forces (or stress T) will be a maximum along lines passing through the point of application of the load and making angles of 45° with the vertical; except in the immediate neighbourhood of the load, it is probable that the ordinary theory expresses almost correctly the distribution of this stress. The vertical compression, N_y, at any point of the beam is determined by a similar assumption to that made above for N_x.

Professor Carus Wilson's experiments were made on beams of glass suitably loaded, and depended on the optical property of glass, that when it is strained, it produces a change of phase in the undulations of polarised light passing through it, sensibly proportional to the strain. These experiments are quoted at length with diagrams of the results, and it is shown that they bear out the theory of the paper.

C. F. F.

Displacement-Diagram for Framed Structures in space of Three Dimensions.

By A. Hübner.

(Der Civilingenieur, 1893, p. 377.)

The Author represents the displacements of all the joints or "nodes" of a three-dimensioned framework structure by vectors drawn from a fixed point or "pole." The displacements may be due either to an assumed displacement of one or more parts of the structure, or to the elastic alterations of length of the members under different conditions of loading. The vectors are represented by plan and elevation, the planes of projection being the same as used to represent the structure. The method is an extension of Williot's method for the similar but simpler problem of a framed

structure lying in one plane, and gives a simple graphical solution of the problem of the changes of form of an elastic framed structure in space.

The method is illustrated by the full discussion of two examples, viz., a pavilion roof with eight members, and a framework dome octagonal in plan.

A. S.

Calculation of the Forces tending to Deform Metallic Tubular Bridges. By — Chartiée.

(Revue générale des Chemins de fer, Sept. 1893, p. 118.)

The Author states that in the study of tubular bridges, that is to say, of metallic works in which the principal girders are held together by top and bottom cross-girders, the forces resulting from deformation under the action of the dead and live load are often neglected. These forces attain such a value, in consequence of the feeble resistance of the upright girders and flooring girders under rails or highways to flexion, that it becomes necessary to determine them as exactly as possible. This determination can be effected with sufficient accuracy by the aid of the method the Author here describes, which he states offers the advantage of being simple.

The section of the two upright girders and the flooring girder of each of the panels of a tubular bridge can be considered as forming only a single piece if the rigidity of their framing is sufficient. The overhead cross-girder, having only to resist buckling in addition to its own weight, may have its resistance to deformation neglected without sensible error. The tangents at the points of junctions of the neutral fibres of the upright and flooring girders ought to form in each case a right angle invariably, whatever the deformations of the upright and flooring girders may be. It is on this hypothesis that the Author proceeds to prove the following formulas:

$$F = H \tan d = -(f + d) \quad . \quad . \quad . \quad . \quad (1)$$

where F = the horizontal displacement at the junction of the neutral fibres of the upright and overhead girders (named K), H = the distance between the centres of the top and bottom girders, d = the angle of rotation between the normal and displaced positions of the girders due to deformation caused by the loads. Then, supposing that a horizontal force Q is applied at the point K to bring it back to its normal position, it would generate in the flooring girder a moment of flexion M = − Q H, and under the action of this moment the flooring girder turns through an angle β, causing an equal angle between the tangents of the neutral fibres of the upright and flooring girders at their point of junction, and the point K to travel through a distance d,

which is equal to H $\tan \beta$; then $F - d = f =$ the remaining horizontal displacement of the point K from its normal position. It will be seen that the force Q can only be applied through the overhead girders.

$$f = \frac{M h^2}{3 E I_m} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where E = the coefficient of elasticity of the metal, I_m the moment of inertia of the upright girder, and h the height from the top of the flooring girder to the point K.

$$d = H \cdot \frac{M l}{E I_e} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

I_e being the moment of inertia of the flooring girder, and l being half the length of the flooring girder.

$$M = - \frac{F}{\frac{h^2}{3 E I_m} + \frac{l H}{E I_e}} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

$$M = - \frac{S}{\frac{h^2}{3 H} \times \frac{I_e}{I_m} + l} \quad . \quad . \quad . \quad . \quad . \quad (5)$$

where S = the surface of the positive moments of the dead and live loads acting on the flooring girder.

$$- Q = \frac{M}{H} = - \frac{S}{\frac{h^2}{3} \times \frac{I_e}{I_m} + l H} \quad . \quad . \quad . \quad (6)$$

The resistance to buckling would be determined by Rankine's formula

$$P = \frac{R \omega}{1 + 0 \cdot 00012 \frac{L^2 \omega}{I}} \quad . \quad . \quad . \quad . \quad (7)$$

in which P is the total pressure supported by the piece,
ω its section,
L its free length,
I its smallest moment of inertia,
R the maximum work of the metal per unit of surface.

Generally the rigid connection between the upright and flooring girders necessitates a joint immediately above the flooring girder at a point where the bending moment is at a maximum; although the presence of gussets compensates partly for this joint, it is to be feared that the section of the upright girders at this point may be subjected to forces surpassing the limits of safety. An

elevation of the joint of the flooring girder to the upright girder is given, the design of which the Author states will ameliorate this disposition without inconvenience in construction, and which shows a continuity of the flooring girder with the upright girder.

It is to be understood that independently of the resultant forces due to the continuity of the upright and flooring girders, the upright girders must be calculated to resist the action of the wind.

Besides the elevation referred to, the Paper is provided with several diagrams.

J. A. T.

Lifting-Bridge at Dijon.

By — Galliot.

(Annales des Ponts et Chaussées, August 1893, p. 261.)

This Paper contains an exhaustive account of the construction of a lifting-bridge over the Burgundy Canal at Dijon to replace an old stone arch, which was inconvenient both for the canal and the road traffic. A controversy which lasted many years took place on the question of whether the new bridge should be fixed or movable, the arguments on both sides being very fully presented in the Paper, and an account being given of the various plans put forward. The plan eventually adopted was that of a lifting-bridge, which, giving when in place a clear headway of 7 feet 10 inches, would pass a great part of the barges without being moved, and which could be raised through a height of 4 feet 3 inches. The estimate for it was £2,600. The span of the bridge is 32 feet 2 inches, giving a waterway of 20 feet, with two towing paths.

The chief interest of the work lies in the method adopted for moving the bridge. A hydraulic press is placed under each of the four corners, and the pressure-pipes are led to a compressing-cylinder with a loaded ram. The load on the ram is such that the bridge can raise it when falling to its seat. When the bridge is to be raised, water is run into a tank also carried by the ram until the weight is sufficient to lift the bridge. The motions of the four lifting-rams are equalized by pinions, one of which is fixed to each ram, and which gear with fixed racks; the four pinions being all connected together by shafting and gearing. The strength of this machinery is made such that if any one of the four rams or pressure-pipes failed, the other three would be able to lift the bridge. Between the compressing-cylinder and the four pressure-pipes is placed a "distributor," with valves to prevent any return flow while the bridge is lifting. By this means no accident to the bridge would follow from a pressure-

pipe or ram giving way during the operation of lifting. A pressure-gauge is fixed to each pipe, visible easily to the attendant. A pump is provided, to be operated by eight men, by which, if necessary, the bridge could be lifted without using the counter-balancing weight. The weight of the compressing-ram and its load is taken by two beams fixed in the walls of the building, when the ram is at the top of its stroke—that is, when the bridge is in its place—so that the pipes are not under pressure except when the bridge is raised. A small pump, that can be worked by one man, is provided, by which the compressing-ram can be raised from its supports so as to remove them, and which also serves to make good any leakage.

The pressure-pipes are taken across the canal in a gully formed in the bottom, covered with iron plates, so as to be accessible.

This bridge, like many canal bridges in France, is paved with old colliery ropes, both on the footways and roadway. They are flat Manilla ropes of about 1½ inch in thickness, and 7 inches and upwards in breadth, laid transversely on the wooden flooring and nailed down. This paving is found to wear well, and to give an excellent foothold to horses in all weathers. The objection to it for movable bridges is that its weight varies greatly, by reason of the moisture it absorbs.

The Paper is accompanied by calculations of all parts of the bridge and machinery in detail, and by detail drawings in four plates.

C. F. F.

Reconstruction of a Bridge over the Seine by the Western Railway of France. By R. Audra.

(Le Génie Civil, vol. xxiii., 1893, p. 297.)

This railway crosses the Seine no less than five times between Garenne-Bezons and Rouen. The bridges were originally of timber, but about thirty-five years ago they were reconstructed with cast-iron arches. One of them was destroyed in the war of 1870, and afterwards rebuilt with straight girders. The others have been a source of anxiety almost ever since their construction, and particularly of late years with the increase in rolling loads; the spandrils have cracked in many places, the cause of which is assigned to the torsional stresses arising from the transverse bracing between different ribs.

A careful study of the structures has led to the decision to rebuild the whole series of arched bridges, and the present article describes the reconstruction of the first, recently opened for traffic. It is called the "Pont du Manoir." The first idea was to replace the cast-iron arches by wrought-iron arches, but the condition of the foundations was such that it was considered dangerous to disturb the condition of loading of the piers to the extent which

such an operation would have involved. The next solution was to replace the arches by girders on the same piers, and to reconstruct each bridge in halves, so as to have always one line of rails available for the traffic. It was found, however, that this plan would require six years for completion, even if more than one of the bridges was taken in hand at the same time. It was considered inadmissible to hamper the business of the railway for so long a period, and another plan was considered, viz., that of making a temporary bridge, so designed as to serve for each structure in turn during reconstruction. This was found to be more costly than building new bridges altogether, and, after certain difficulties about deviating the line had been overcome, the plan of making entirely new bridges was decided on.

At the "Pont du Manoir" the old bridge was in six spans, while the new one is in three spans, with a total width between abutments of 668 feet. The foundations were laid by compressed air on a chalk foundation, the greatest depth below low-water in the river being 42 feet. The girders are double-webbed lattice girders 26 feet 3 inches deep, 29 feet 6 inches apart between centres. The cross-girders are 3 feet 3 inches deep and 14 feet 7 inches apart, with trimmers carrying a floor of $\frac{5}{16}$-in. flat plates. The cross-girders are attached to the lower boom, and the main girders braced overhead. The girders are continuous over the piers, and were erected by launching out from the abutments. The stresses during erection by this method were greater than they will be in use under the greatest rolling-load. The bridge cost £80,100, independently of deviations, &c., and of the demolition of the old bridge.

C. F. F.

Displacement of the Old Girders of the Isar Bridge at Kleinskal.

By — LUCAS.

(Der Civilingenieur, 1893, p. 490.)

The single-line Altpacka-Reichenberg railway (Austria) crosses the Isar by a seven-spanned viaduct, each span being 26 metres (85·3 feet). The old girders had to be replaced by wrought-iron girders of more modern construction, and during the work the traffic was not to be interfered with. The change of the girders was performed for one span at a time. A scaffold on each side of the bridge was prepared, the new girders mounted on the right scaffold, and the left being ready for the reception of the old girders which were moved sideways at a favourable opportunity.

The whole work of displacing the old girders, putting the new in position, and replacing the rails ready for traffic occupied ninety-six minutes, reckoned from the time of passing of the last train.

A. S.

Improving the Navigability of a River by Training.

By Professor H. ENGELS.

(Der Civilingenieur, 1893, p. 554.)

This Paper was read at the International Engineering Congress at Chicago, on 3rd August, 1893.

The improvement of a navigable river by "training" or "regulation" is limited to the execution of works to guide or control the channel, so that its configuration may be maintained by and when subjected to the scouring action of the water. In this sense the Author excludes from discussion in the Paper, canalization, dredging, or any other method in which the natural scour is not the active agent employed. The mouths of rivers are not discussed in the Paper.

The acceleration of the water due to its surface fall is partly spent in overcoming frictional resistance due to the roughness of the river bed. If along any stretch of the river the acceleration is exactly neutralized by the resistance, the water flows with uniform velocity. If the river bed be not uniform, part of the energy of the moving water is expended in altering its form. In a river bend, the water, in consequence of centrifugal force, is driven against the concave bank, and the channel at this side is gradually deepened and extended. A condition of stability of form is reached when the resistance of the channel against washing away is equal to the effort of the moving water. The former, depending on the material forming the channel, remains constant, while the latter is lowered by a reduction of velocity. This reduction of velocity follows from the deepening and extension of the concave side of the river bends, and the consequent increase of the length of the river. Should the quantity of water flowing down-stream be increased, the form of the channel will no longer be stable and changes will take place. The duration of flood-water is, however, never long enough for a new form of stability to be attained. Silt, loosened from the bed and banks, is carried down stream, the heaviest pieces rolling along the bottom, the lighter being carried in suspension. When the flood subsides, the lighter portion of the silt is deposited on the convex sides of bends and the heavier the concave sides, since the agitation of the water is greater on the concave than on the convex sides.

There are thus three characteristic parts into which a river can be divided; an upper part in which the erosion of the channel is not yet completed, a middle part in which erosion and deposit are equal, and a lower part in which the deposit exceeds the erosion. In a river stretch in which the erosion is completed the channel is of settled form, and the water moves over its own alluvial deposit; while in a stretch in which erosion is not completed the channel is unsettled and gets gradually deeper. In the latter case any silt brought down must travel the full length of the

stretch, and will be gradually reduced in size during the transport.

It is of course impossible to make direct observations on the transport of silt by rivers; but it may be observed, for example, that a shoal either retains its place, or after some time is swept away. If a new shoal appears about the same time lower down the stream, before it can be said that the shoal has merely changed its position, it must be proved that the second shoal is made up of the same materials as the first was. The Author made some experiments for the purpose of elucidating this point. A channel of sheet zinc was made in the hydraulic laboratory of the Technischen Hochschüle at Dresden, and filled with sand, the particles of which were of uniform size but differently coloured. Water being allowed to flow through the channel, it was found that when erosion took place the particles of sand travelled the whole length of the apparatus. With a smaller slope of the channel, the sand particles experienced a small movement down-stream, but in a short time complete equilibrium of the sandy bed was established. A greater flow of water being permitted, another movement of the sand took place until a new form of equilibrium was established. The characteristic of a river with settled channel is that with the normal flow of water the form and size of the channel is stable, and only a partial disturbance of equilibrium takes place in time of flood. No training of an unsettled channel should be attempted. Training works can only be successful, as regards duration, in settled channels.

In conclusion, the Author says:

1. Only river-stretches in which natural erosion has been completed are suitable for training-works. River-stretches with still unsettled channels can only be improved for navigation by canalization.

2. The equalization of the surface fall at low water can only be obtained by training-works, on stretches of uniform regime and having the same condition of bed throughout.

3. This equalization of the surface fall, in a favourable case, can only be obtained when the possibility of erosion is prevented by the building of walls along the banks above low-water level.

4. As regards the attainment and continual preservation of this equalization, after the building of the banks, the bottom of the river should be secured where subjected to erosive attacks by the water.

Mere regulation of the width of the river cannot produce the highest possible degree of navigability.

5. The greatest possible depth at low water may then be determined by the equations

$$Q = b\,t\,v$$
$$v = K\sqrt{t\,J}$$
$$t = \left(\frac{Q}{b\,K\sqrt{J}}\right)^{\frac{2}{3}}$$

where t is the attainable depth, v the velocity, Q the quantity of water discharged, b the breadth at low water, J the surface fall, and K can be approximately determined from the second of the equations.

A. S.

Improvement of the Lower Weser. By P. T. Kapteyn.

(Tijdschrift van het Koninklijk Instituut van Ingenieurs, 1892–93, p. 183.)

Since the summer of 1887 very important works have been in execution between Bremen and Bremerhaven, on the Lower Weser, under supervision of the Weserkorrektion administration. The undertaking consists principally of fascine-dams, of a total length of 25 kilometres, while the total quantity of material employed since 1888 amounted to about half a million cubic metres annually.

The works may be described as bulkhead-dams, stretch-dams, groynes, and submerged groynes. At formation level, above the usual high water, the dams are 13 feet 4 inches wide, with slopes on both sides of 1 to 1, on a foundation mattress, with a lap or apron on each side of 33 feet. The groynes are located where the river-current requires deflecting. The labour is contracted for, but the Direction furnishes all materials.

These consist principally of brushwood and rubble stone. All wood is accepted except poplar, delivered at the works tied in fagots of different dimensions, and mostly utilized immediately on being discharged from the ships, which bring large quantities from great distances. Besides the mattresses built on the spot, a great many osier-rafts of different dimensions, mostly 1 metre thick, are built floating, and afterwards sunk in the required positions. The brushwood, conveyed on the spot, costs on the average 1 mark 60 pfennige, or 1*s.* 7½*d.* a cubic metre, and a cubic-metre mattress-work costs 1 mark for labour.

The rubble, or riprap, stone is obtained from near Osnabrück, and costs on an average 7·55 marks per cubic metre, or, roughly, about 16*s.* per ton delivered on the works.

Although the administrator is responsible for the materials, all labour, plant and dwellings for the labourers must be provided for by the contractors.

The mattresses are, as a rule, of smaller dimensions than those usually made in Holland, but of greater thickness. Instead of willow bands and straps, galvanized wire is mostly used, which, it is said, gives a greater lateral strength. A mattress sunk in position costs, in round numbers, about 3 marks per cubic metre, or 2*s.* 3*d.* per cubic yard.

H. L.

Erosion of Banks of the Mississippi and Missouri Rivers.

By J. A. Ockerson, M. Am. Soc. C.E.

(Transactions of the American Society of Civil Engineers, June 1893, p. 396.)

A general survey of the Mississippi river from Cairo, Ill., to Donaldsonville, La., was made by the Mississippi River Commission during the years 1879 to 1883. This comprised a system of secondary triangulations by means of which numerous stone monuments were located at frequent intervals along the river to serve as initial points for future surveys. Additional sets of stone survey-marks were fixed, indicating sections of the channel at intervals of 3 miles along the river; each set consists of four stones, two on either side of the river at ⅓ mile and ⅝ mile from the banks.

In the autumn of 1891 the condition of these stones was investigated, coupled with a redetermination of the bank lines in the bends; a length of 885 miles of the river was thus examined. The Author compares these two surveys together and with the sketch map made by Captain Philip Pitman in 1768. The most striking features in this comparison are the remarkable stability of the larger islands and the persistency of the sharp bends, even though the entire bank line may recede several hundred feet. For instance, Profit Island, the last one passing down the river, 815 miles from Cairo, occupies the same position now as when Captain Pitman passed it a hundred and twenty-five years ago; and Coles Point, a narrow neck of land, finally cut through in 1884, had stood for over a hundred years as a barrier, forcing the river 6 miles to the eastward almost at right angles to its general course, and although many attempts had been made at different times by means of large ditches to effect this result, it was finally cut through at an entirely different place.

Huge trees that have withstood the floods and the encroachments of the river for several scores of years testify that, during the time of their growth, the peregrinations of the river have been confined within comparatively narrow bounds.

No great amount of caving occurs at high water, the period of greatest activity in this respect is generally found to be during the falling stage. The banks, composed of horizontal layers of sand and clay at irregular intervals, become saturated with water for a distance from the river, during the high-water period, and it flows out as the river falls, carrying with it large quantities of the sand. This action, aided by the impinging currents in the bends, causes large blocks of ground, sometimes 200 feet or more in width, to settle down bodily several feet. This mass is finally broken up and disappears; cut-offs are generally occasioned in this manner.

Excessive curvature and erosion are seldom coincident, and the latter seems to be independent of cultivation. The maximum

amount of caving is found to be coincident with the levees, but there does not appear to be any relation of cause and effect between them. The maximum rate of erosion between the surveys of 1883 and 1891 was at a distance of 620 miles below Cairo, and amounted to 224,000 square yards per annum per mile of the caving length.

The total area of caving per annum, calculated from the same data, is 39,000,000 square yards, or 43,000 square yards per mile of river. The average annual amount of erosion of the right bank per mile of river is 21,700 square yards, and of the left bank 21,400 square yards.

The annual volume of erosion per mile of river is 972,000 cubic yards. It may be generally accepted that erosion of one bank is followed by an accretion of the other; sometimes the latter exceeds the former. The amount of silt in suspension in the river at New Orleans accounts only for one-third of the erosion, the remainder going to form the accretions and the bars.

The Author further compares the above conditions of the Mississippi with those which obtain in the Missouri.

A. W. B.

Construction of the Graving-Docks Nos. 5 *and* 6 *at Havre.*

By Maurice Widmer.

(Annales des Ponts et Chaussées, June 1893, p. 1061, 8 plates.)

Previous to the year 1889, there were but four graving-docks at Havre, and of these only one was sufficiently large to admit ships of any considerable size, such as the steam-ships belonging to the General Transatlantic Company, the Messageries Maritimes, the Pacific Company, the Havre Peninsula Company, the French Commercial Steam-ship Company and the United Freighters, all of which start from Havre, or the many large foreign steam-ships which have frequented the port in increasing numbers since the Bellot basin and dockyard were opened in 1880. The want of increased accommodation had long been felt, and another graving-dock was commenced twenty years ago, adjoining the largest of the four above-named, but it was not proceeded with, and the definitive studies for Nos. 5 and 6 were not begun until the year 1884. No. 5 was completed in October 1889, and No. 6 in May 1890.

The supporting blocks in Nos. 1, 2, 3, and 4, are respectively 147 feet 8 inches, 201 feet 9 inches, 259 feet 11 inches, and 426 feet 6 inches in extent. Of these four docks, the first three were constructed in 1871–72, at the same time as the Citizen basin into which they open, and No. 4, which is the largest, in 1863–64. The latter is at right angles to the large basin of the Eure, with which it is connected by a sluice 84 feet 6 inches long and 98 feet 6 inches wide; and Nos. 5 and 6 have been placed parallel to it and between No. 4 and the canal from Havre to Tancarville, with gates and sluices into the basin of the Eure, of the same length as No. 4 but

narrower, being respectively 65 feet 6 inches and 53 feet in width. The length to which the supporting blocks extend is 377 feet in No. 6 and 492 feet in No. 5; and, in order to admit any of the large steam-ships of the Transatlantic Company of the most recent construction, four of which are 508 feet long, a semi-circular prolongation at the further end, beyond the blocks (similar in form to that adopted in the other docks), makes the entire length of No. 5 537 feet. The blocks are on a horizontal plane, differing in this from the system more usually adopted at some other ports, where they are longitudinally inclined.

The four earlier docks were built with concave floors and are drained by two channels placed on either side of the blocks, but, following the example of Antwerp, Nos. 5 and 6 are constructed with a slightly convex bottom, so that any rain, or other water, which may penetrate to the dock, or which may be thrown from ships when emptying their water-ballast while in dock, runs off to the sides, and thence through a number of glazed earthenware tubes, placed 6 feet 7 inches apart, into culverts built at a lower level beneath the side-walls, in which there is room for it to accumulate, before being pumped off, to the extent of 22,000 cubic feet without the blocks being wetted. These culverts are connected with centrifugal pumps, worked by four steam-engines, which exhaust the water from the dock. The height of the side walls to No. 5 is 23 feet 7 inches above the floor, and the width of the dock is 59 feet at bottom, increasing by successive steps, about 8 feet high, to 90 feet at the surface, each of these steps forming a terrace-walk ranging from 2 feet to 3 feet 4 inches in width, and the highest being on a level with the quay.

The soil to be excavated consisted of a layer of clay mixed with peat, and below that a layer of sand, another of gravel, and then a bed about 10 feet in thickness of very compact black pebble which is found in the district at nearly the same depth in and around the port of Havre; beneath which is a bed of fine sand of apparent indefinite thickness. The volume of excavation for the two docks amounted to 233,450 cubic yards, and 26,215 cubic yards of concrete were used in the work.

The black pebble forms the foundation of both the docks; upon it was first laid a bed of concrete, made of three-parts flint pebbles and two of mortar composed of sand and Portland cement, and then a brick pavement 9 inches thick. The side-walls are faced with brick, 2 feet 9 inches thick, with a solid backing generally about 3 feet 4 inches in width, of siliceous limestone blocks; and the successive steps are topped with granite slabs, 3 feet 4 inches long by 8¼ inches thick.

Dock No. 4 being already provided with the requisite pumping machinery, the culverts which surround Nos. 5 and 6 were extended as far as the engine-house of No. 4, the machinery of which has been increased and arranged so as to exhaust the water from the three docks, by the addition of a smaller supplementary draining-well, 3 feet 9 inches deeper than that of No. 4. Both during the

construction of these works and subsequently, the organization of the pumping-arrangements offered some unexpected difficulties, and the connection of the several culverts to allow the three docks to be drained separately, involved some delicate operations, requiring the use of caissons and compressed air, which are fully described by the Author. Some troublesome infiltrations of water were effectively stopped by injecting Portland cement.

The engine-power had to be much augmented, for it used to require from nine to eleven hours to empty dock No. 4 with an engine of 75 HP., driving two powerful centrifugal pumps, in addition to two smaller force-pumps; whereas it was considered necessary to have power sufficient to empty the dock in three hours and a half. With this view it was decided to substitute for the old machinery three principal engines and two smaller ones, and, projects and tenders having been invited from the chief engine-makers in France, the preference was given to the Forges et Chantiers Co., of Marseilles, who proposed for the principal work three centrifugal pumps with a separate condensing-engine to each, and three boilers, each capable of supplying steam to either of the three engines, placing bronze turbines, 6 feet 7 inches in diameter, at the bottom of the main draining-well; besides two smaller pumps in the supplementary well for drawing the water from the lowest level of the culverts and keeping the docks dry. It had been specified that two of the larger engines, working together, should suffice to empty No. 4 dock in three hours, and when tested on three successive days, they did so in from two hours forty-six minutes to two hours fifty-six minutes; the total power exerted having been rather more than 600 HP., and the volume of water raised to a height of 28 feet 4 inches rather more than 8,000,000 gallons.

In the four earlier graving-docks the usual wooden blocks for supporting the ships are employed, but in Nos. 5 and 6, the example of Liverpool and Antwerp has been followed and the blocks are of wrought iron, shaped as shown in one of the accompanying plates, it being found that they are more conveniently shifted when examining the ships' bottoms.

The chief items of expenditure in constructing and fitting Nos. 5 and 6 were as follows:

	£.
Earthworks and masonry	129,752
Portland cement	20,560
Lime	5,618
Engines, pumps and engine-house	18,498
Sluice-gates	9,839
Blocks and other ironwork	2,999
Paved road round the docks	4,000
Railing round the docks	800
Cranes	1,992
Sundries	20
Total cost of docks Nos. 5 and 6	194,078

The tonnage of vessels which entered the Havre graving-docks in 1888 and 1889 was respectively 351,035 tons and 396,412 tons; which, after the completion of Nos. 5 and 6, rose to 473,812 tons in 1890, and to 531,404 tons in 1891; the dues paid in the latter year amounted to £12,643.

O. C. D. R.

New Locks of the St. Denis Canal. By MAURICE RENAUD.

(Annales des Ponts et Chaussées, July 1893, p. 44.)

The St. Denis Canal connects the basins of La Villette (Paris) with the Seine at St. Denis. It is 4⅛ miles in length, with a total fall of 93 feet. The deepening of the Seine to admit vessels of a draught of 10 feet 6 inches necessitated a similar deepening of the canal, and at the same time the state of disrepair of the locks made a reconstruction of them desirable on this ground. In the old canal there were twelve locks, having a width of 25 feet 6 inches: its depth was 6 feet 7 inches. All the locks and the greater part of the bridges have been rebuilt and the depth has been increased to 10 feet 6 inches.

The canal is frequented by two classes of boats—those from the northern canals which are small, and those from the Seine which are larger. With a view, therefore, to economise water the locks have been made double, one set being of the same size as those of the northern canals, with a chamber 126·3 feet by 17 feet, and the other having a chamber 205 feet by 27 feet, to accommodate the larger craft. With this arrangement the advantage may be obtained of concentrating all the machinery on the island between the locks and so leaving both banks free. In order to obtain this advantage to the full, all the lock-gates have been made in one leaf only. The culverts for emptying and filling the locks are contained in the masonry of the island.

In the reconstruction the number of locks has been reduced to seven, two pairs of adjoining locks having each been replaced by a single lock, and a set of four locks having been replaced by a single lock of 32 feet 6 inches fall.

The reconstruction was rendered somewhat difficult by the fact that navigation could only be suspended during a period of two months, and also because the neighbouring land was so far occupied that in some cases a temporary deviation of the canal was impossible.

The gates are of iron framework covered on the upstream side with wood planking. They close (in the manner usual in France) against a plane surface, instead of making a watertight joint in the hollow quoin, the heel post being clear of the masonry. They are moved by hydraulic power from a turbine, a quadrant being attached to the gate and connected by spur gearing with a hori-

zontal shaft from the turbine. This same shaft transmits power to all the four gates of one pair of locks. The method adopted at the first pair of locks rebuilt was that of using a straight arm hinged to the back of the gate and driven in or out by an endless chain. This plan was unsatisfactory owing to the sudden variations of effort which caused frequent breakages of the chain.

The valves used for the filling and emptying of the locks are all of the cylindrical balanced type, in most cases 4 feet 7 inches in diameter. The lock of 32 feet 6 inches fall was necessitated by the conditions of the site, which only afforded one place where a new lock could be built and no ground available on the banks. To have reconstructed the old locks on the same site would have put a stop to navigation for a longer time than was permissible. A boat-lift would also have occupied too much space.

The objections to deep locks are three: (1) the great size of the gates required, (2) the great consumption of water for locking, (3) the hindrance to traffic by the time occupied in locking.

The first objection was met in this case by closing the upper part of the lock at the lower end with a masonry bridge giving the same headway as other bridges on the canal. The gate closes on an upper straight sill in the masonry of this bridge similar to the lower sill, and is thus supported on all four edges. This bridge carries a road over the canal. The second objection was met by forming two reservoirs, one in connection with each lock and of the same size as the lock. The bottom of the reservoir is at 10 feet 10 inches above the lower water-level, so that, in locking down, one-third of the water discharged is turned into the reservoir, and again returned to the lock when locking up. The third objection is largely met by having more and larger orifices of discharge. A higher speed can be allowed with a deep lock as soon as the vessel has a considerable depth of water under her, because less disturbance will be caused by the entering water than in smaller depths. Speeds of 1 inch to 1¼ inch of rise per second have been given without any inconvenience.

The walls of a lock of this height require great consideration because of the frequent and extreme variations in load to which they are subject. In this case the wall has been so built that the water-pressure will be entirely transmitted, it is supposed, to the soil behind the wall. The wall is 19 feet 8 inches in thickness and 45 feet 8 inches in height. The back of the wall, as well as the face, is vertical, and the section is reduced towards the top by pockets in the middle of the wall. Thus the upper part consists really of two walls, one facing the water-pressure and the other the earth pressure, the two being connected by cross walls. The excavation was made so as not to disturb the soil in any way, the face being carefully boarded and strutted. A wall of the same weight and of ordinary construction would not have been regarded as safe.

The cost of the works has been as follows:—

For a lock of 7 feet 6 inches fall, replacing a single lock, £20,000;

for a lock of 14 feet 9 inches, replacing a pair of smaller locks, £34,000; for a lock of 32 feet 6 inches, replacing four locks, £74,000.

The Paper is accompanied by Tables of the time occupied in opening and closing gates, in emptying and filling locks, and in making the passage of the canal, and is followed by an appendix containing detailed calculations of strength of all parts of the gates. It is illustrated by five plates, and also by diagrams in the text.

C. F. F.

Inclined Plane for Canal-Boats at Beauval.

By T. A. Mallet.

(Mémoires et Compte-rendu de la Société des Ingenieurs-civils, May 1892, p. 627.)

The inclined plane described has been made to connect the Canal de l'Ourcq with the River Marne, where the difference of level is 40 feet. Locks would have been too costly and would have drawn off too much water from the canal.

The boats transferred are 92 feet in length, 10 feet beam, and 4 feet deep. The weight, empty, is about 16 tons, and loaded 70 to 75 tons.

Power is derived from a turbine established at a weir on the Marne, and forming part of a general system of power-distribution for the district. The head of water at the weir is 5 feet 3 inches.

Any barge to be transferred is brought on to an iron truck, carried by two four-wheeled-bogies of 6 feet 6 inches wheel-base. The wheels of these bogies are made with a central flange, so that they can run either on rails outside or inside the flange. This enables a device to be used by which the barge remains horizontal in rising out of the water at the commencement of its journey, and in sinking into the water at the end of its journey, in spite of the opposite inclinations on which it is travelling in the two cases. The slope of the railway at the upper end in rising from the canal is 6 per cent., and after crossing the canal-bank it runs down to the river on a 4 per cent. gradient. While the barge is clear of the water, only one pair of rails is used, and the barge has the same inclination as the railway; but at the ends of the line an extra pair of rails is laid, on which, in one case, the forward truck, and in the other, the rear truck, is made to mount by means of the double-tread, so as to bring the barge on an even keel. The power of the turbine is conveyed to the truck by rope-transmission, the rope running at 50 feet per second, and being ½ inch in diameter. Its motion is reduced by gearing and transmitted to a pinion which engages with a rack in the same fashion as is common on mountain railways. The journey is 492 yards in length, and occupies, in practice, about thirty-five to forty minutes.

Brakes, controlled by a governor, so as to act automatically, are provided.

The total cost was about £4,000, including everything, except motive-power and the branch canal made to bring the boats to the foot of the inclined plane.

C. F. F.

The Reclamation of the Pontine Marshes. By Luca Rossi.

(Giornale del Genio Civile, 1893, p. 265.)

The resanitation and repopulation of the Roman Campagna—deserted for centuries past on account of the pestiferous exhalations due to imperfect drainage and the evaporation of stagnant water from the undulating volcanic soil—was one of the earliest objects of consideration of the government after the unification of the Kingdom of Italy: the first Royal Commission for the preliminary study of projects bearing date October 20th, 1870. The scheme took definite shape in the law of December, 1878; and the first portion dealt with was a tract of nearly 5,000 acres of the miasmatic marshes centering in the Stagno di Ostia, and the enclosing belt of higher ground.

This basin, fringing the ancient but now disused Southern arm of the Tiber estuary, on which stood the city of Ostia (the small hamlet of modern Ostia being a short distance further inland), extends eastwards as far as the range of hills from the Dragoncello to Castel Porziano, and southwards to Tor Paterno; while it is severed from the sea by a range of sandhills, through which the ancient channel—the *forma emissaria*—finds its precarious and changing way. The area of the low-lying tract may be classified as follows:—

	Acres.
a. Permanently submerged	1,690
b. Inundated in winter	768
c. Saturated and useless	2,008
d. Higher ground, fairly dry	227
	4,693

or about 7⅓ square miles.

In planning the drainage works for effecting the reclamation, one principal object was to separate those portions from which the water would flow by gravitation, from the lower portions from which the water would have to be raised by mechanical means—so as to reduce the latter work to a minimum. This lower ground forms the central area of the system, so that the water from the eastern slopes has to be carried on a raised channel across the mere to join the high level intercepting drains of the western

sandhills. The lengths of the several high-level main drains are as follow:—

	Feet.
1. Dragoncello (north-western area)	13,803
2. Lingua (north-eastern area)	16,454
3. Pantanello and Tor Paterno (southern area) .	15,322
4. Ostia (western area)	14,469
5. Castel Fusano (south-western area) . . .	16,333
6. Connecting channel (east to west)	2,607
Total . . .	78,988

The general inclination assigned to these main channels was 1 in 3,330; but the connecting channel was 1 in 5,000. The total length of this drain is 2,607 feet, of which 2,139 feet are in raised channel. Considerable difficulty was experienced in the construction of this earthwork upon the yielding waterlogged base, and subsidence amounting to an average of 4 feet continued for nearly five years before a condition at all approaching stability was attained.

The high-level drains vary from 11·5 feet to 6·56 feet above the sea.

The chief constructive difficulty was experienced in connection with the stability of the slopes. The originally projected $1\frac{3}{4}$ to 1 was found to be quite insufficient, and the proportion was soon altered to 2 to 1, and, in some cases, $2\frac{1}{2}$ to 1. In the low-level outlet the slope had to be set back to as flat as 4 to 1.

The old *forma emissaria* was re-opened, with due study of the prevailing winds and consequent silting tendency. The original bed is very irregular, varying at first from 80 to 100 feet in width, but narrowing to about 30 feet towards the outlet, and finally dispersing in small and shifting channels until lost in the sand. It is intended to extend the new channel to a sufficient distance beyond low-water line, between suitable moles; but no ultimate provision has yet been made for this portion of the work.

The drainage of the low-level area (4,693 acres) comprises a network of open drains and trenches, spaced at an average distance of 550 yards, and with an average bottom width of 3 feet 3 inches, all contributory to two main drains running from N.W. and from S.E. ends of the marsh respectively. The average surface-level of the furrows and hollows is 2·62 feet above sea-level. Allowing for subsidence by drainage to the extent of 0·65 foot, and the necessity of lowering the surface of the subsoil water to a further 2·62 feet below the ground-level, so as to admit of proper cultivation, and with an ordinary depth of 2·30 feet of water in the drain, the level of the bottom of the drain should be 5·57 feet below the first-mentioned level, or 2·95 feet below sea-level.

The low-level outlet basin is 1,070 feet in length, with a bottom width of 54 feet, and slopes originally standing at $1\frac{1}{2}$ to 1. The upper surface is a stratum of compact clay, and this rests on a

thick peat bed; so that when the excavation laid this surface bare, and the lateral pressure was removed, the lower bed was distorted and forced up. The work was renewed repeatedly, but continually forced up again; so that the bed was raised nearly 18 inches, and the work was interrupted for a considerable length of time. The channel-bed was finally increased in width to 65 feet, and the slopes set back to 4 to 1, at which point stability was secured. This basin or channel terminates at the pumping-station, where the water is raised 8 feet 3 inches, and discharged by a basin 64 feet in width and 8 feet 3 inches in depth, to join the gravitation waters of the high-level system.

The building comprises a central block (engine-room) 65 feet 6 inches by 39 feet 9 inches, and two wings: the total length being 189 feet. The engine-room rests on a solid concrete foundation, 74 feet 10 inches by 43 feet 8 inches, at a depth of 29 feet 6 inches.

There are two engines, each 39 HP., and the boilers are of Lancashire type, with Galloway tubes. Each boiler is 22 feet 11½ inches in length, and 6 feet 6¾ inches in diameter: the total heating surface being 753 square feet (each). The turbines (Pillon) are four in number, viz., two of 61·27 inches in diameter, making 131 revolutions per minute, or a periphery velocity of 2,095 feet per minute; and two of 39·24 inches in diameter, working at 249 revolutions, or a periphery velocity of 2,550 feet per minute.

The engines were started in December, 1889, with water-level at about 12 inches over the marshes; and after one hundred and sixteen hours of intermittent working this level was lowered all over the marsh to the extent of 19½ inches. Following this, the preliminary hindrances being surmounted, a brief period of work brought the level down 6 feet lower (*i.e.*, the total lowering being from + 30 centimetres to − 2 metres). The average working-period is twelve hundred hours per annum.

The cost of the work may be stated as follows:—

		£.		£ per Acre.
a. Land-compensation and occupation works		15,393	or	3·28
b. Drainage-works, culverts and outlet basin, and pumping-station		45,712	„	9·74
c. Machinery		3,603	„	0·76
	£.			
Annual working expenses	1,500	64,708		13·78
„ maintenance charges	715			
	2,215, capitalised	= 44,300	=	9·44
		Total	. . .	23·22

The reclamation of the whole area has now been effected; the district is becoming repopulated, and plentiful crops of wheat, barley, and oats are raised upon the land.

P. W. B.

Earth-Subsidence caused by boring an Artesian Well at Schneidemühl. By H. Chudzinski.

(Centralblatt der Bauverwaltung, 1893, p. 277.)

This Paper gives an account of the boring of a well at Schneidemühl, which resulted in such serious subsidences of the ground that numerous houses fell and others had to be pulled down.

The boring was begun in an old well and ultimately reached a depth of 73 metres (239·5 feet). To a depth of 9 metres (29·5 feet) gravel was met with, then layers of clay and clayey sand of different thicknesses. At 50 metres (164 feet) water was met with, but not suitable for domestic use; the boring was therefore continued to a depth of 70 metres (229·6 feet). Water was then struck which rose through the bore-hole with great violence, bringing with it considerable quantities of mud. An overflow was fixed to the lining tube 5 metres (16·4 feet) above ground level. As the water did not cease to flow, the tube was driven 3 metres (9·8 feet) deeper, and when this too proved unavailing, and as the water also rose outside the lining tube, the latter was drawn, and an attempt was made to plug the hole by means of narrow bags filled with sand and clay.

This also proved futile, and it was decided to build a well of masonry of 2·70 metres (8·8 feet) internal diameter, which was to reach the upper layer of clay and to rise 6 metres (19·6 feet) above ground level. The progress of this well was, however, so slow that other steps had to be taken. In order to be quite sure of the formation of the ground, a second bore-hole was made to a depth of 25 metres (82 feet) and 4·5 metres (14·7 feet) distant from the original one. A shallow one was also drilled to observe the level of the ground water. Then a tube 19·5 centimetres (7·67 inches) in diameter was driven into the old bore-hole to a depth of 11 metres (36 feet) when the spring in the clay was tapped. Then a second tube was sunk in the immediate vicinity to a depth of 13·5 metres (44·3 feet) and connected with the older tube, so that the water from the one could be driven into the other. Within the new tube a second one was inserted, the lower part of which was perforated with round holes for a length of 5 metres (16·4 feet) from the bottom. The diameter of this inner tube was 15 centimetres (5·9 inches), and it was driven to a depth of 45 metres (147·6 feet).

Large stones were now ejected, and on one occasion a clay plug 1 metre (3·28 feet) long was washed out from the bore-hole. A wooden plug was then fastened to the top of the tube, and upon this an 8-centimetre (3·15 inches) tube was fixed. An opening was left 5 metres (16·4 feet) above ground level, which had a good effect; but it was decided not again to close the bore-hole altogether until the earth movements had ceased. It was calculated that from May 4th to June 21st, the well yielded 132,000 cubic metres (29,040,000 cubic gallons) of water, carrying with it 5,800 cubic metres (7,586 cubic yards) of earth. W. F. R.

Sinking Shafts through Quicksand. By Peter Jeffrey.

(Journal of the Illinois Mining Institute, 1893, p. 91.)

Large and valuable deposits of coal, iron ore, and other minerals frequently occur overlaid with glacial drift of various depths, consisting of clays, quicksands, and other matter; and in some localities it is necessary, in order to reach the mineral, to sink shafts through these unstable beds, an operation which often entails a large expenditure of capital and labour. The Author briefly notices some of the more prominent of the numerous methods of sinking shafts through quicksand which have been tried with more or less success.

The first of these methods is the Poetsch freezing system, which consists in freezing the running sand to a solid mass, to an extent much larger than the area of the proposed shaft, and then blasting out the sand in the same manner as in ordinary rock. This system has been very successfully adopted in parts of Europe under difficult circumstances; and, in the Author's opinion, it should rank as one of the best, if it be not the only system, which can be successfully applied where the sand is of great depth, contains water, and lies between strata of rock. Instances are cited by Mr. R. de Soldenhoff in a Paper that appeared in the American *Colliery Engineer* of March, 1888, which showed that the Poetsch system was very effectual but at the same time very expensive, the cost of freezing alone amounting to from £60 to £176 per yard in the cases described. The sand in these instances ranged from 18 feet to 100 feet in thickness, with overlying strata from 15 feet to 200 feet.

Another method which has been very generally used is the caisson, or drop-shaft, which may be successfully adopted where the quicksand extends from a point at or near the surface down to slate or hard pan. One serious objection to this method of sinking through quicksand, especially in localities where the sand is deep, is that the great pressure to which the walls of the shaft are subject may prevent the sinking of the caisson as rapidly as the material is removed from beneath the shoe, and thus permit the sand to run down behind the casing and into the shaft, necessitating the removal of a much larger quantity of material than would be contained within the area of the shaft itself. The in-flow of sand in some cases causes a subsidence of the surface for a considerable distance around the shaft, requiring in some instances the removal of the hoisting engines further away from the shaft, and the building of cribs to support the tracks for the transportation of the excavated sand. Meanwhile, operations in the shaft have also to be suspended, and sometimes sand and water may run in and fill up the shaft nearly to the surface. All this may occur several times before the solid measures are reached, and shafts sunk by

these means are seldom perfectly plumb, and often require re-timbering in order to put them in workable condition.

A third method to which the Author calls attention consists in the use of an improved caisson or crib, patented by John McGovern, of Illinois, in the year 1885. This caisson is constructed of boiler-plate and wood, and is arranged to be sunk in advance of the permanent timbering of the shaft. An illustrated description of the contrivance and the method of sinking with it in sand or mud is given, and the Author summarises the advantages of the plan over the ordinary caisson system as consisting, (1) in preventing the rushing in of the sand beneath the timbers of the shaft while new timbers are being set, and, (2) in permitting the application of jack-screws to force the shoe continually downward, thus preventing delays and keeping the work under constant control.

The caisson may be made of any desired length, and need be kept only a sufficient distance in advance of the excavation to prevent the in-rush of sand.

The discussion of this Paper elicited a statement from a Mr. Rice to the effect that the McGovern system had been used with entire success in the sinking of the shaft of the Whitebreast Fuel Company at Ladd, Bureau County, Illinois. This shaft was 155 feet deep, and penetrated a very troublesome bed of quicksand containing boulders, which made it exceedingly difficult to manage the descending frame. Several unsuccessful attempts had previously been made by the ordinary method.

A. H. C.

On the Methods of Investigation to establish the Self-Purifying Power of River-Water. By THEODOR KÖHN.

(Deutsche Vierteljahrsschrift für öffentliche Gesundheitspflege, 1893, p. 693.)

The German Chancellor was unable to direct that a systematic investigation should be carried out on all the watercourses in the kingdom liable to be polluted by sewage, as recommended by the Public Health Society, at their meeting in September 1891, and this task must therefore be undertaken by those most interested, namely, by the towns polluting any watercourse. The Author points out that it would be advisable to lay down some definite system upon which an examination of this kind should be conducted. The town of Charlottenberg, before introducing a plan of sewage-irrigation in the year 1890, as also during 1891 and 1892, directed investigations of this nature to be undertaken as in the case of the River Spree. By reference to a plan, the Author shows the points selected for the collection of samples, the mode of procuring the latter being as follows:—The river, being embanked, is kept to a uniform breadth of 50 metres, and at each of the three sampling stations ropes were drawn across the stream at right angles to the

banks. By means of five knots in the rope, the total span was subdivided into six equal spaces, and at each knot samples were taken, at a uniform depth of 1 metre below the surface. To carry this out an empty stoppered-bottle, attached to a marked iron rod, was plunged into the river to the required depth, when the india-rubber stopper was withdrawn by means of a string; the bottle was then raised to the surface and securely corked. This work was entrusted to Dr. Knöfler, who carried out also, in his laboratory, all the chemical tests. The reasons for the selection of the test-places are discussed, and it is pointed out that in the case of samples taken in this way, with all due precaution, the variations observed in the different parts of the river, at the same station, are sometimes greater than those at two stations a considerable distance apart. To prove the accuracy of this statement, a Table of analyses is given, which sets forth, in detail, the composition of the samples of river-water at each section of each of the selected stations, as also the results of the bacteriological tests. In a second Table a collective review is given of all the observations, extending over a period of more than two years. The Author states concerning these latter observations, that they tend to show that it is a difficult matter to formulate conclusions respecting the pollution of rivers by the introduction of town-sewage, and concerning the self-purifying power of streams, by investigations of this character. But it may be regarded as a fact, that self-purifying powers are possessed by rivers, and that these are due, partly to mechanical, partly to chemical, and partly to biological processes. The Author appends certain proposed rules for systematic investigations of this nature, based upon the results obtained in the course of this inquiry.

G. R. R.

Investigations into the Pollution of the Rhine by the Cologne Sewage-Water. By — STEUERNAGEL.

(Gesundheits-Ingenieur, 1893, p. 473.)

Attention is directed to a recent bacteriological investigation into the condition of the Rhine, conducted by Drs. Stutzer and Knublauch, at the instance of the municipal authorities of Cologne. The researches of Professor v. Pettenkofer into the self-purification of the Isar, and of Dr. Schenk into that of the Rhine, are quoted. A brief account is given of the stretch of the River Rhine, which formed the special subject of this inquiry, and which extended from above Cologne to the vicinity of Düsseldorf, a flow of about 49 kilometres (30·4 miles). This portion of the river is shown by means of a map, and a diagram is likewise given to illustrate graphically the degree of self-purification attained. The first large town situated on the river above Cologne is Bonn, 27 kilo-

metres (16·7 miles up stream), with a population of forty thousand, whose sewage passes direct into the Rhine. It would seem, from the bacteriological tests, that, in the flow between Bonn and the outskirts of Cologne, the river-water regains its normal condition, and that the condition of the Rhine at Marienburg, adjacent to Cologne, resembles its state before passing Bonn. From Cologne onwards the pollution caused by the population on the left bank of the river is not very important, but on the right bank are several large towns discharging their sewage into the stream. Thus, directly opposite to Cologne is Deutz, with twenty-one thousand five hundred inhabitants; 2 kilometres lower down is Mülheim, which has a population of thirty-one thousand, and which, jointly with Kalk (twelve thousand inhabitants), passes its sewage into the river. Further down are several smaller towns, and the River Wupper, which is highly polluted and which conveys the dye-waters of Barmen and Elberfeld and the sewage from an industrial population of upwards of a quarter of a million persons, enters the Rhine about 20 kilometres (12·4 miles) below Cologne. The pollution caused by other small tributaries and townships is of less importance. The researches of previous investigators, Messrs. Plagge, Proskauer, Prausnitz and Frank, are discussed, and certain of the results obtained by Mr. Frank and Mr. Prausnitz are set forth in a tabulated form. Some of the figures given by the latter are said to be remarkable, in that they indicate a very rapid diminution of the number of germs in the course of a short flow of the Isar.

In all, eight stations were chosen for the collection of test-samples of Rhine water, due weight being given to the various incidental sources of pollution before alluded to, and, during the period from April to November 1892, some six hundred specimens, from the right bank, the left bank and the centre of the stream, were bacteriologically examined. The precautions taken to secure reliable and uniform results are described. It was necessary to undertake the examinations of the samples in a perfectly fresh condition, but a certain interval of time was needed for transport. As it was found, by repeated observations, that in six hours the number of bacteria present in any sample of river-water was, as a rule, doubled, the tests were in all cases made six hours after collection of the sample, the figures obtained being then divided by two. The results, which are set forth in two voluminous Tables, show that a steady improvement took place as the river, which had been considerably polluted by the Cologne sewage, flowed onwards towards Düsseldorf. This improvement was, however, seriously affected by the inflow of the foul water in the Wupper; but notwithstanding this, the Rhine water at Volmerswerth, 41 kilometres (25·4 miles) below Cologne, had almost entirely regained its original condition.

The Author discusses the causes which influence and bring about the apparent self-purification of streams, as evidenced by the decrease in the number of bacteria present. These results are

partly due to simple subsidence, but they are partly also caused, as Professor v. Pettenkofer has pointed out, by the plant-life present in the water. This being the case, attention has been directed to the algæ, and especially to the water bacteria and the *Beggiatoa alba* found in the Rhine. A noteworthy feature in the present figures is the very rapid decrease of bacteria near the sources of pollution and the subsequent gradual and steady diminution in the more remote reaches of the river; for this it is impossible to offer any explanation. In conclusion, some figures are given to show the dilution that the Cologne sewage undergoes in mixing with the Rhine water, as also the total solids present in a given volume of river-water before and after passing Cologne. The proportion of sewer-water to river-water is approximately as 1 to 1,960.

G. R. R.

On Speed of Filtration. By G. Oesten.

(Gesundheits-Ingenieur, 1893, p. 505.)

The answer to the question, What is the speed of filtration? is apparently so obvious—namely, that it is the hourly rate at which the water on the filter-bed sinks in the basin in consequence of the permeability of the layer of sand—that such a query seems to be a needless one. A rate of 100 millimetres an hour, a maximum, and a normal rate are spoken of, and the idea always conveyed is that of the ratio of the superficies multiplied by the sectional area of the water above the sand F to the hourly yield of the filter-bed Q, from which gives V = the filtering speed, or $V = \frac{Q}{F}$.

This speed is spoken of with certainty as a constant one, and it is deemed to be such, so long as the same filter regularly continues to yield a uniform volume of clarified water. In fact, as F is a fixed quantity, V must, it is thought, be constant, so long as Q is maintained unchanged. In modern works provision is made by regulating arrangements to keep Q constant—of course, within certain limits—and thus, as everybody assumes, arises a constant speed of filtration.

When this question comes to be looked into more carefully, it is, however, found that V does not represent the rate at which the particles of water sink in the open basin, but the rate at which they traverse the interstices of the sand. If the former rate, equal to V, is constant, the latter may fluctuate very considerably, in accordance with the changes in the condition of the filtering material, and may represent very varying values, always greater than V, the apparent filtering speed.

By reference to diagrams of circles, to represent sand-grains in

contact, filling a given area, the Author shows how it is possible to arrange the particles in greater numbers and therefore having less interstices in the same space. In the position first selected, the spaces between the sand-grains, where there are four points of contact to each, are equal to about one-fifth of the whole sectional area of the filter; whereas in the second diagram, where there are but three points of contact between each sand-grain, the capacity of the interstices is approximately but one-tenth of the area of the filter. In the former case, the actual speed in the pores of the filter would be not less than five times the so-called speed of filtration, and in the latter it would be ten-fold that speed. These interstitial spaces are clearly the same, both for small and for large grains of sand. No doubt, in practice, while the sand-particles are not spherical, there are examples of both systems of arrangement; but, at any rate, it would not be safe to regard the actual spaces traversed by the water as greater than 0·2 of the observed area occupied by the clarified water, and thus, in a perfectly clean filter, the actual speed is never less than five times the so-called speed of filtration, and in accordance with the position of the particles (whether with three or with four contacts) this speed must vary in each stratum of the bed. The Author considers the different factors which tend to choke the pores and to clog the filter, and he points out that, in order to maintain a uniform yield, it becomes necessary, as the filter-bed wears out, to increase the head of water above the sand, the result of which is really to drive the water through the diminishing area of the interstices between the sand-grains at a greatly increased velocity. Calculations are given to show how largely this real filtering speed, which is at its minimum, five times the observed speed, may vary at different periods of the working of the beds. From these it is deduced that if the yield is really to be kept constant, there must be no alteration of the head of water at first adopted. The apparent yield will, of course, in this case steadily decrease throughout the life of the filter-bed, and the amount of water clarified will be greatly below that which is obtained at an apparently uniform working speed.

G. R. R.

The Sanitation of Towns. By LOUIS GENIS.

(Bulletin de la Société Industrielle de Marseille, 1893, p. 17.)

The object of this paper is a general examination of town sanitation with special reference to the recently proposed sewage works at Marseilles. When introducing a project of law requiring the Government of the Republic to promote sanitation improvements in France (December 3rd, 1891), the Minister of the Interior referred to the declarations of more than one eminent man in support of his assertion that the public health is a question of

essentially democratic and popular interest, and he declared it to be the absolute duty of a democratic government to initiate, or support, all well-devised plans that tend to the diminution of preventable diseases. But too little attention has been paid to this subject in France, with the result that the average death-rate in towns above 10,000 inhabitants is no less than 25·45 per 1,000, as compared with 19 or 20 per 1,000 in London, Berlin, Brussels, Frankfort, Munich, &c., and 21·16 per 1,000 in the rest of the population of France.

There are no reliable statistics in France, such as exist in other countries, to show the causes of death, doctors being required by Art. 378 of the Code to preserve professional silence on the matter; but from 1886 to 1890 the chief causes of death were specially ascertained in 200 of the large towns, with a joint population of 8,673,489, with the result that 33 per cent. of the deaths were shown to have been due to preventable causes.

Impure water, house-sewage, factory residues and accumulations of street filth are the chief causes of infection, owing to the noxious gases which arise from the fermentation, or putrefaction, of the organic matter they contain, and the hygienic conditions of a town depend upon the rapidity with which these matters are dealt with. This involves, on the one hand, a supply of pure water, and on the other a canalization sufficiently perfect to keep the streets clean and to carry away all kinds of fluid sewage. At Marseilles the house-supply of water for domestic use should not be less than 22 gallons per head of the population; besides $14\frac{2}{3}$ gallons per head for cleansing the streets and $18\frac{1}{3}$ gallons for keeping the high-pressure reservoirs of Realtar and Sainte-Marthe sufficiently full to ensure a sufficient flow of water in the sewage-pipes. Total daily supply, 55 gallons per head of the population; and Marseilles, having, under the proposed project, an estimated provision of 120 gallons per head for 400,000 inhabitants, has decided to double the extent of the existing canalization in order that the water supplied to the houses may in future be kept entirely separate from the existing canals, which are in some parts of the town open to the air and admit foul water from the adjoining factories.

The project which has received the approval of the Municipal Council is to take water for domestic consumption from the canal in the suburb of Four-de-Buze, at a rate of 330 gallons per second, and convey it in pipes 2 feet $7\frac{1}{2}$ inches in diameter, to the reservoir of Sainte-Marthe, which will have a capacity of 68,000,000 gallons, from whence it will be distributed to the houses, through pipes of the aggregate length of $132\frac{1}{2}$ miles. For other than domestic purposes, it is proposed to supplement the present supply of water by pumping it from the sea into a high-pressure reservoir, and the Author answers the criticisms which have been addressed to the use of salt water for such purposes, by quoting from a paper on the subject, by Mr. Cockrill, published in the Minutes of Proceedings of the Institution of Civil Engineers, in 1892, in which

the opinions of eighteen English engineers are cited in support of its adaptability for sewer-flushing and street-watering.

It is not sufficient to bring pure water to the houses unless, after use, it is properly disposed of. At the present time there are still far too many cesspools in which sewage and kitchen-refuse are allowed to accumulate for days and even longer periods, until they begin to putrefy and sometimes overflow, after which the contents are usually removed to central factories where they are chemically treated in order to be rendered profitable, either as manure for the land, or for other purposes. This is a system which has been generally condemned by all who understand anything of hygienic principles. Another system is to convey the whole of the sewage to a distance by natural gravitation, using as little water as possible, and sometimes the solid part of the sewage is separately carried away; but every system which encourages economy of water is to be condemned, for it is certain that proper sanitation requires abundance of water and that all sewage should be quickly removed by absorption with it.

How this can best be effected continues still the subject of as much debate in France as it was in 1880, when the committee on Paris smells (*La Commission des Odeurs de Paris*) reported against the construction of sewers which, in their opinion, would increase the noxious effluvia in the streets; but an accumulation of scientific opinion in their favour is presented, and the practical results obtained since their introduction in London, Brussels, Berlin, Frankfort and other large cities, have conclusively proved that, with a good supply of water, sewers have everywhere tended to diminish the death-rate. They must be constructed with a sufficient fall to keep the liquid sewage in motion, for if the gradient is less than 1 in 5,000, the solid particles which float in the liquid settle to the bottom and form an objectionable deposit in the sewer. With this gradient, however, water should flow, in a well-constructed sewer, at the rate of 1½ mile per hour, which is sufficient to carry off the sewage before it has time to putrefy; but it is safer to give a rather greater fall, and in Marseilles the minimum which has been adopted varies from 1 in 3,333 to 1 in 2,500.

The sewers should be well ventilated, for air hastens the oxidation of their contents, and they must be built with care so as to prevent any possible leakage into the subsoil. According to Pasteur, as soon as the putrefaction of a dead body commences, it destroys the infectious matter which may have appertained to the living person, or animal, and this applies also to the solid excreta in sewage.

Wherever a system of sewers has been introduced it has had the same beneficial effects. In London, where the contents of water-closets are now wholly discharged into the sewers, the number of deaths from typhoid fever per 100,000 inhabitants has diminished since their introduction from an average of 91 (1850–1860) to an average of 26 per annum during the last fifteen years; in Frankfort the average was 68 per annum in 100,000 from 1851

to 1869, and fell to 23 during the years 1877 to 1879, after the introduction of a system of sewers; while in Paris, where cesspools still prevail, the average varies from 50 to 140. For the sake of a miserable economy the owners of houses in Paris limit the supply of water in every possible way, and make the greatest efforts to reduce the number of water-closets, replacing them by contemptible privies which are focusses of pestilence and fill the apartments with suffocating emanations.

O. C. D. R.

Experiments on the Disinfection of Town Sewage with Sulphuric Acid. By Dr. M. Ivánoff.

(Zeitschrift für Hygiene, 1893, p. 86.)

The fact that the cholera-bacillus displays an intense susceptibility to the action of acids was known to its discoverer, for Dr. Koch pointed out that in the acid secretions of the stomach these bacilli speedily lose their vitality. The subsequent experiments of Kitasato have shown that very minute additions of sulphuric and hydrochloric acids to bouillon cultures destroy cholera germs in the course of a few hours; and, lastly, Messrs. Stutzer and Burri have studied exhaustively the effect upon these organisms of very dilute solutions of sulphuric acid. At the suggestion of Professor Pfuhl, the Author undertook to investigate the action of dilute sulphuric acid upon cholera-bacteria when present in sewage water. From the inception of his experiments he surmised that under these conditions the acid must be used in a more concentrated form than in the above cases, because the sewage water invariably contains substances which would combine with the acid, and would thus, to some extent, neutralise its effects. It was, however, ascertained that the additional amount of acid rendered necessary on this account was but trifling. The samples of sewage water were derived both from the Berlin and the Potsdam sewers, and were infected alternately with pure cultures of the cholera-bacillus and with the fresh dejections of a cholera patient. The acid in the case of the Berlin sewage water was used in three degrees of strength: (1) as a 0·02 per cent. solution; (2) as a 0·04 per cent. solution; and (3) as a 0·1 per cent. solution. The conditions under which the analyses were made are fully described, and the Author availed himself of microscopic observations, as well as bacteriological tests. Four parallel series of experiments were carried out, and the results were in every case identical. The Potsdam sewage, which is three times as concentrated as that of Berlin, was treated with stronger acid, viz., with 0·04, 0·06, 0·08, and 0·12 per cent. solutions. It was found that in the case of the Berlin sewage the 0·04 per cent.

solution of acid was fatal to the cholera-bacilli, but that with the stronger sewage water of Potsdam, the amount of acid needed was that present in the 0·08 per cent. solution. It is pointed out that, whereas previous to treatment the sewage water was faintly alkaline, the sample to which 0·08 per cent. of acid had been added had a strongly acid reaction on being tested with litmus paper, and that such reaction may be regarded as an indication that the necessary dose of acid has been employed. This treatment is, with the exception of the use of lime, the cheapest system that can be adopted.

G. R. R.

The Newer Developments of Towns from a Sanitary point of view. By A. Frühling.

(Der Civilingenieur, 1893, p. 435.)

This is a lecture delivered before the Sachsischen Ingenieur- und Architeckten-Verein. The Author refers to the modern tendency of the population of a civilized country to collect in large towns, and the consequent necessity for the preservation of the public health of carrying out extensive works of sanitation. Water-supply, the drainage of the soil, the sewerage system, the disposal of sewage, good paving and thorough cleaning of the streets, are discussed in turn.

For the future expansion of a town, the Author suggests that the municipality should prepare plans by which its growth, instead of being allowed to take place in a haphazard fashion, would be regulated, and a rational provision made for increase of traffic, development of trade and industry, and the erection of factories and dwellings. The erection of hospitals, schools with gymnasiums and playgrounds, baths, and the laying out of public parks, are referred to. The problem of smoke-prevention is also touched upon. With regard to the cost of these public works, the Author estimates that the diminution of loss of money and working time through sickness much more than repays the original expenditure.

A. S.

Experiments Concerning the Action of Peat-dust upon the Bacteria of Cholera and Typhoid Fever.

By Professor Dr. C. FRÄNKEL and Dr. E. KLIPSTEIN.

(Zeitschrift für Hygiene, 1893, p. 333.)

The problems involved in the safe, speedy, and economical removal of human dejections can by no means be regarded as solved at the present time, though it is now conceded that the claims of the agriculturist can only have weight when the conditions laid down by the sanitarian and the physician have been complied with. From the hygienic point of view it is imperatively necessary that all excrementitious matters, as also domestic refuse and waste water of every description, should be so dealt with as to destroy the germs of infectious diseases, and from this aspect the water-carriage system is the only one that can be regarded as in any sense a complete plan of sewage-disposal. Even this method of dealing with waste matters has defects to which the Authors allude. There are, however, doubtless cases where this system of sewage-removal cannot be carried into practice, and where it becomes necessary to have recourse to some other plan of treatment. In recent years the employment of powdered peat in dry closets has found many advocates, for by this means the urine is absorbed, the solid dejections are deodorized most effectually, and, moreover, the resulting compound is one which can very readily be conveyed away and rendered available for agricultural purposes. In fact, the use of peat in this manner would have become still more widely extended, but for the fears expressed in certain quarters that this substance when so employed would intensify the danger of infection, because the porous peat would be likely to store up the pathogenic germs and serve as a carrier of diseases. From certain researches by Schröder, it, however, became evident that, in addition to its properties as a disinfectant, peat possessed the power of destroying the cholera-vibrios, though certain other species of bacilli proved themselves more capable of resisting its influence. In consequence of the inquiries set on foot by the German Agricultural Society,[1] many investigations into the action of peat have recently been undertaken, and the Authors have conducted a series of tests with two descriptions of peat, one of which had a strongly acid reaction, the other was more feebly acid. Samples of each kind of peat, both sterilized and non-sterilized, were added to cultures of cholera-, typhoid fever- and other germs, and the results are set forth in tables. It is deduced that in from 2½ to 5 hours the comma bacillus is destroyed by the addition of peat-dust; but exposure to this substance for only one half or one hour will sensibly diminish its vitality. The cholera-vibrios may

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[1] Minutes of Proceedings Inst. C.E., vol. cxiv. p. 430.

retain their vitality for as long as fourteen days in a mixture of peat-dust with urine; but, as a rule, they are destroyed at the end of a week, or in very acid urine even after only one day. Mixtures of peat with various substances valuable in agriculture (kainit and superphosphates) were also tested by the Authors; kainit was found to be wholly devoid of influence upon the cholera bacteria, while both kinds of superphosphate examined proved themselves to be possessed of marked effect in augmenting the germicide powers of peat. Investigations, specially undertaken to determine upon what properties in the peat this action depended, demonstrated that its influence was due to the acids therein contained, and led to the conclusion that in order to increase the efficiency of the peat to the utmost, care must be taken to intensify as far as possible this acid reaction by artificial means. Numerous Tables are given to show the effect of various mixtures of peat, urine, and superphosphates upon bacteria; and the Authors state that the theory can no longer be maintained that peat possesses marked preservative powers with respect to the germs of infection, and that, on the contrary, there is reason to believe that peat exerts a considerable disinfecting action, which may be intensified by the addition of suitable materials, and even increased to very notable proportions. Under all circumstances, where a dry system is unavoidable, as, for instance, in the case of isolated buildings, hospitals, barracks, &c., the use of dried peat may be confidently recommended as furnishing a safe, cheap, and reliable system of dealing with excreta.

G. R. R.

Classification of Wheel-Motors on the basis of the Method of Action of the Water. By Ugo Ancona.

(Der Civilingenieur, 1893, p. 359.)

The different methods by which water can be made to perform mechanical work are: 1st, by its weight; 2nd, by shock, *i.e.*, when a stream of water impinges at right angles on a moving surface; 3rd, by "action" or "impulse," *i.e.*, when an unconfined stream of water meets a moving surface, the relative velocity having no component at right angles to the surface, glides along and ultimately leaves the surface; 4th, by reaction, *i.e.*, when a stream of water enters, flows through and ultimately leaves a moving pipe or channel, which it completely fills; 5th, by a combination of two or more of the above methods of action.

A water-motor working by weight alone is not possible, since the water must enter and leave the buckets with some definite velocity. A motor working by shock alone is possible, but this is such an inefficient mode of action that it should be avoided. A number of existing types of turbines are pure "impulse" motors.

A pure "reaction" motor is not possible, the so-called reaction turbines acting both by reaction and impulse.

The classification of existing types of motors is as follows :—

WATER WHEEL-MOTORS.

Water Wheels.	*Turbines.*

Water Wheels.

Axis horizontal; partial admission; great dimensions; small velocity. The water acts either by weight or by shock, as under—

By Weight.	*By Shock and Weight.*	*By Weight and Impulse.*
Undershot water wheel.	Overshot and breast wheels; bucket wheel.	Poncelet wheel; Zuppinger wheel; Sagebien wheel.

Turbines.

Axis in any position; complete or partial admission; small dimensions; great velocity. The water acts neither by weight nor by shock.

Reaction Turbines.			*Impulse Turbines.*		
The water works simultaneously by reaction and impulse.			Pure impulse is possible.		
Axial Flow.	*Radial Flow.*		*Axial Flow.*	*Radial Flow.*	
	Inward.	*Outward.*		*Inward.*	*Outward.*
Henschel-Jonval turbine.	Francq turbine.	Fourneyron, Cadiat, Segner, and Scotch turbines.	Girard turbine, Hänel turbine, Pelton wheel.	Tangent wheel.	Girard, Hänel, and Schwam-Krug turbines.

The fundamental equations for all the principal types of motors are given. Diagrams showing the division of the total available head into pressure-head, velocity-head, and head lost in friction at any point of the water's passage through the motor accompany the Paper.

A. S.

On the Employment of Metal Sleepers. By A. FLAMACHE.

(Bulletin de la Commission Internationale du Congrès des Chemins de Fer, 1893, p. 179.)

The Author states that all engineers have been struck with the extreme divergence of opinion which exists between railway companies on the subject of the employ of metal sleepers. On the one hand, the English companies, the great majority of the French, the Belgian State, and in general the railways of the west of Europe, are all systematically opposed to the use of metal sleepers. On the other hand, metal sleepers have enthusiastic supporters in Germany and Austria, where the engineers consider the question as solved.

There are, however, some singular facts in this divergence of opinion. Systems of line which are not able to withstand the traffic for eight hours on an English line (such, for example, is the Haarmann rail) have been tried and have lasted for many years on certain German lines. Also whilst some declare that the cost of maintenance of the metal sleepers is much less than that of wooden sleepers, others declare that the cost is from 2½ to 3½ times that of wooden sleepers.

The Author proceeds to point out that it is the very high speeds that so rapidly disperse the ballast around metal sleepers, and that the ballasting itself is one of the chief difficulties to be overcome in the use of the hollow sleeper. The metal sleeper ought to rest on the ballast with a plane surface, or at least a surface which does not require ballasting in the hollow. A new sleeper, the Boyenval and Ponsard, has been brought into use in France which is rapidly extending there, and which has features which overcome to a great extent the objections of the ordinary types. It is formed of a double trough, the sides of which have only a small inclination, and this, combined with the comparative smallness of the hollows, forms a kind of filling up which moves in every sense with the sleeper, and the ballasting in the troughs does not reunite with the adjacent ballast. It results that this profile is practically the equivalent of a sleeper resting with a plane surface. Apart from this trial of the Boyenval and Ponsard sleeper, the Author does not know of any serious endeavour to realize the planeness of the surface on the ballast, he eliminating as impossible the sleepers formed of an iron U laid down.

The Author then takes out the moments of the forces acting on the sleeper, and in conclusion states that the sleeper for great lines of railways can only be acceptable under the following conditions:

1. It ought not to require packing in the hollow, that is to say, that it ought to be able to displace itself longitudinally and transversely, as a sleeper in wood. Its form ought to be strictly prismatic.

2. It ought to have a weight of 165 to 176 lbs., and be made of steel, in order to offer a moment of resistance of 5,400 to 5,800 foot-pounds. Iron, insufficiently tenacious, in the form of U placed either face upwards, unfavourable to resistance to flexion, ought to be excluded. The profile ought to be symmetrical from the point of view of flexion.

3. The resistance ought to be obtained from the full profile, that is to say, that the clipping hinges should be excluded.

4. The attachment ought not to permit of rectangular punched holes, but only of drilled ones, of a preference round.

The Paper is accompanied with sketches of the Bernard, Severac, and the Boyenval and Ponsard sleepers.

J. A. T.

The Cost of Maintenance of Metal Sleepers. By — Ast.

(Bulletin de la Commission Internationale du Congrès des Chemins de Fer, 1893, p. 17.)

The Author states that the Administration of the Belgian State Railways has been compelled to replace a great number of metal sleepers of the Post and Braet types after a short period of trial of five years, and in spite of the enormous sums relegated to the maintenance of the line. These unfortunate experiences are so much the more to be regretted as the faults of the Vignoles rail with wooden sleepers are well known, and the necessities of an economical working attract from day to day more attention on the metal superstructure, notably from the point of view of reducing the cost of the running maintenance.

Contrary to the results of these experiences, which have lasted five years, the Emperor Ferdinand Northern Railway has obtained, during a period of observation of nine years, very satisfactory results in the employ of metal sleepers of the Heindl type. These have been placed in a trial-section in a manner admitting of being compared with wooden sleepers during the same length of time, under the same conditions. But in spite of very heavy traffic the Heindl sleepers have remained intact during this period of nine years, whilst the oak sleepers present traces of wear corresponding to the tonnage and to their age. A Table is given showing the cost of maintenance of the line laid with sleepers of the Heindl type, and also of the line laid with wooden sleepers, for each year, from 1884–1892, the average annual cost for that period of the metal sleepers being £27 10*s.* 2*d.* per mile, and of the wooden sleepers £34 16*s.* 5*d.* per mile. Another Table gives all the particulars of the rails, sleepers, and ballast used in conjunction with the Post, Braet, and Heindl types of sleepers, which shows comparatively little difference between them. The Author states, therefore, that the failure of the metal sleepers used by the Belgian State Railways can only be attributed to faults in the details of construction, and notably in the mode of attaching the rails, or in the employ of unsuitable materials. In that which concerns the former the Author finds in the direct connection of the rail to the sleeper a principal cause of its rapid destruction. This means of attachment transmits all the forces directly on to the sleeper in the most unfavourable conditions. It follows that a slackness is soon produced, which, under the influence of the dynamic forces, increases rapidly and occasions finally the effects of hammering which are transmitted to the ballast and destroy it.

On the Emperor Ferdinand Northern Railway they have found on the trial section of the Heindl type no slackness of the means of attachment nor any injurious wear. There is also an absence of evil effects on the sleepers and ballast. The system of the attachment of the Heindl type possesses, in the Author's opinion,

all the advantages which the inventor endeavoured to obtain. The insertion of a supporting-plate protected against displacement on the sleeper, avoids all immediate mechanical effort on the body of the rail, as much on the means of attachment on the exterior side of the rail as on the sleeper itself. Not only does the supporting-plate protect the sleeper and the means of attachment against the friction of the rail, but it acts as a buffer in the transmission of the shocks exerted on the sleepers. Owing to the ingenious construction of the fastenings, the vertical and horizontal forces are separated, and attain the sleeper under the most favourable conditions.

The following Table gives the annual cost of maintenance per mile of the different types of sleepers used on the Belgian State and Emperor Ferdinand Northern Railways:—

Type of Sleepers.	Wages.	Materials (Ballast excluded).	Ballast.	Total.
	£ s. d.	£ s. d.	£ s. d.	£ s. d.
Belgian State Railways (mean of 5 years).				
Oak sleepers	4 8 7	6 5	nil	4 15 0
Metal sleepers of Post type .	43 8 6	2 2 9	47 0 2	92 11 5
„ „ Braet type .	34 2 3	2 11 8	52 9 4	89 3 3
Nord Empereur Ferdinand Railway (mean of 9 years).				
Oak sleepers	27 13 6	6 14 7	8 4	34 16 5
Metal sleepers of Heindl type	25 14 3	1 5 9	10 2	27 10 2

A Table is also given showing the number of hours devoted to the packing of ballast, and the amount of defective material withdrawn from the line, on a section of observation half a mile long, between Antwerp and Brussels, laid with oak sleepers for a period of five years.

The great difference in cost of maintenance between the Post and Braet types and wooden sleepers on the Belgian State Railways is due to the large amount of attention which the ballasting is constantly requiring by the use of these types of metal sleepers.

In addition to the Tables referred to a plan and sections of the sleepers of the Heindl system and their attachments are given at page 176.

J. A. T.

Wooden Plugs of Oak or Fir Creosoted for filling up the Holes of Treenails, &c., in Sleepers under Traffic. By — THIRIET.

(Revue générale des Chemins de fer, August 1893, p. 97.)

These plugs were formerly roughly manufactured by hand by the plate-layers during the enforced leisure of rainy days; they were, however, found to be unsatisfactory, owing to the variation of size and form, as well as to the waste of wood. Mr. A. Collet has invented a machine which is provided with knives that cut the wood, at the same time preserving the fibre, in such a manner that the plugs which it produces can be struck without fear of breaking. The octagonal prismatic form given to the plugs by preference, is that which is mostly used; the angles of the octagon impress themselves progressively in the wood, lessen the shock of sudden insertion, prevent the splitting of the sleeper, and oppose themselves to the rotation of the rail-fastenings if any come at the same place.

The absence of a sharp point ensures stopping where the holes are not pierced through. This machine lends itself equally to the manufacture of plugs of square sections—sometimes employed for pine sleepers—and even to rectangular sections for the holes of old caulkings.

The Orleans Railway and a part of the Paris, Lyons, and Mediterranean Railway use four millions of these plugs annually, at a cost of 7*s.* 3*d.* a thousand.

They are employed both on main lines or on branches and sidings. The results obtained by the preservation of the sleepers and the adherence of the attachments of the rails have always been excellent, and such sleepers as were put aside have been rendered again available after plugging.

The wood from which these plugs are manufactured is always chosen from the best parts of old sleepers. If the holes of sleepers are not well stopped up a zone of rottenness is produced which rapidly spreads.

The Author concludes by stating that round plugs are to be avoided, inasmuch as he has found that they have not been able to fill the hole at the bottom, and following this fault the sleepers have been attacked by rot.

The Paper is accompanied by sketches of the plugs made by the machine, and of the rot in the sleeper produced by badly stopped holes; and a plan and elevation of the machine is given in the succeeding number of the "Revue" (p. 144).

J. A. T.

Permanent Way with Spring Seatings and Supported-Fish-Plate Joint. By J. SCHULER.

(Organ für die Fortschritte des Eisenbahnwesens, 1893, p. 184.)

The Author states that the chief disadvantage of the ordinary method of fastening the rails to the sleepers is that a too rigid connection is made which tends to increase the wear and tear, as well as to produce hard running.

Where timber sleepers are employed the Author uses a cast bed-plate screwed to the sleepers, and beds the flat-bottomed rail upon a steel spring laid between two cast clamps; the latter are fitted into the bed-plate and hold down the rail by pressing upon two steel springs laid upon the lower flange of the rail and interlocked with the clamps. The clamps are secured to the bed-plate by bolts which also adjust the pressure of the springs upon the flange of the rail. Where steel sleepers are used no cast bed-plate is required, but the general principle of the seating and clamps is maintained, although they are slightly modified to suit the altered conditions.

The cast bed-plate is $12\frac{13}{16}$ inches long by $3\frac{1}{2}$ inches broad, varying from $\frac{7}{16}$ inch to $\frac{11}{16}$ inch in thickness to give the necessary inclination to the rails, and is screwed down to the sleeper by three screws. On each side of the rail the bed-plate is raised, forming a hollow with inclined edges of sufficient width to receive the steel-spring seating and a protruding under-lip on the inner edge of each of the clamps; these raised portions are $2\frac{1}{4}$ inches long, and have rectangular holes, $1\frac{1}{2}$ inch by $\frac{7}{8}$ inch, cast in them to receive the clamps and clamp bolts.

The clamps are exactly similar to each other, and consist of a base-plate $2\frac{3}{8}$ inches wide and $\frac{1}{2}$ inch thick, with an inner protruding under-lip whose outer face is vertical and inner face inclined, so that the lip fits into the space left on the outside of the rail in the hollow portion of the bed-plate, and with an outer protruding under-lip whose outer face bears against the outer edge of the rectangular hole in the raised portion of the bed-plate; and they have also a hook overhanging the flange of the rail, under which are placed the upper springs. The bolts which secure the clamps are $\frac{17}{32}$ inch in diameter.

The spring seating under the rail is a curved steel plate, 7 inches long and 4 inches wide (the width of the bottom flange of the rail) in the portion which is situated between the clamp plates, widening out to $4\frac{7}{8}$ inches, so as to be keyed on each side by the clamps. Its thickness is $\frac{13}{32}$ inch, and its pitch in the loose condition is $\frac{1}{8}$ inch.

The steel springs, which are fitted between the clamps and rail-flange, are of the same length and elevation as the seating spring, and they are also keyed on one side in a similar manner by the clamps as is the seating spring: they are $\frac{15}{16}$ inch wide at the centre, and on the centre line of this portion for the full length of

the spring a small semi-circular beading is formed on the upper surface to interlock with a groove in the hook of the clamp-plate in order to prevent lateral movement of the spring.

The springs are put under an initial pressure of 1 ton when the clamps are bolted down, this having been determined by experiments made by the Author with a view to obviate the danger of the springs working loose. He has also found that a wheel load of 10 tons is required to cause the maximum deflection of the spring.

As before stated, the Author has endeavoured to make the elasticity of the rail uniform throughout, and at the rail-joints this can only be accomplished by making a connection which will prevent any lateral or vertical deviation of the rail ends from a true line and which will not be too rigid. Such a connection would also lessen the wear and tear of the rolling stock and prevent any undue shocks, which the Author considers is not attained by the ordinary methods of making the rail-joint.

The Author has experimented with various forms of rail-joints, and has finally selected the form hereafter described, and which has been made use of by the Baden State Railway Administration on the Mannheim and Basle line. Two fish-plates on either side of the joint are used, the inner one being $22\frac{5}{8}$ inches long, and the outer one $7\frac{13}{16}$ inches long. The inner plate fits between the head and flange of the rail, the centre portion being cut away at the top in order that a more direct support to the heads of the rails may be given at their ends by the outer plate which is designed to fit into this recess; this centre portion of the inner plate overhangs to a depth of $2\frac{1}{2}$ inches below the underside of the rail, and immediately under the ends of the rails a rectangular hole $2\frac{5}{16}$ inches by $1\frac{1}{2}$ inch is made to receive a triple-divided key which rests in these outer plates and so freely supports the ends of the rails on their undersides. The inner plate following the contour of the bottom flange of the rail forms an inclined plane upon which is supported the base of the outer plate, and the tightening of the bolts which pass through the two plates causes a firmer support to be given by the outer plate to the heads of the rails.

The weights of the spring seatings, &c., are given in detail, and amount to 7·23 lbs. each for steel-sleepers, and to 14·87 lbs. each for timber-sleepers. The weight of the fish-plates and their connections at the rail-joint amounts to 68·12 lbs. The total track weights, excluding the weights of the rails and sleepers, are as follows:—

——	Steel Sleepers.	Timber Sleepers.
	Lbs.	Lbs.
For a 9-metre (= 9·84 yards) rail with eleven sleepers	300	$463\frac{1}{2}$
For a 12-metre (= 13·12 yards) rail with fourteen sleepers	345	553

The Paper is provided with a plate giving full details of the seatings and rail-joint arrangements. J. A. T.

Mechanisms Employed for the Starting of Compound Locomotives. By A. Lavezzari.

(Mémoires et Compte rendu de la Société des Ingénieurs-civils, October 1893, p. 329.)

The Author recalls attention to the Papers on compound locomotives by Mallet, Polonceau, Pulin, Lencauchez, &c.

The starting mechanism almost always consists of apparatus for the admission and cut-off of steam, interposed between the high- and low-pressure cylinders, and serving to make the low-pressure admission and the high-pressure cut-off independent of each other. The starting apparatus of M. du Bousquet on the Northern Railway is described, which enables the two groups of cylinders to be controlled independently of each other and worked either as compound or as all high-pressure. Those of the Baudez on the P. L. and M. Railways have been described in the "Revue Générale des Chemins de fer."

The Author thinks, however, that, for goods trains especially, a simpler system is required, and that the greater risk of total disablement in case of an accident to one cylinder might be faced. It is explained that, with valves so arranged as to permit an admission of steam up to 90 per cent. of the stroke, the intermediate apparatus is useless, because the action of the cylinder valves of itself distributes the steam in the requisite way. Further, that by means of orifices in the valve seat of the low-pressure cylinder, boiler steam can be admitted direct to this cylinder automatically, the admission only taking effect when the admission port is open for more than 50 per cent. of the stroke. This condition would only arise at starting.

The first locomotive on which this principle has been applied is a goods engine of the Austrian State Railways with two cylinders, and it gave remarkable results both as regards starting facility and power. It draws regularly, on a gradient of 1 per cent. 8½ miles long, trains of 570 tons, ordinary engines of exactly the same class (except as regards the cylinders) only drawing 460 tons.

The engine-driver accustomed to simple engines requires no special instructions. The Austrian Government is constructing several engines of this type.

C. F. F.

Locomotive Valve-Gear with Separate Admission- and Exhaust-Valves. By E. Polonceau.

(Annales des Mines, vol. iv. 1893, p. 525.)

The Paris and Orleans Railway Company have from time to time tried to get more economical results from their locomotives than is possible when using the ordinary slide-valve for both admission and exhaust. In 1858 a Meyer expansion valve-gear was tried, but the results were not satisfactory; the increased economy obtained being more than counterbalanced by the extra friction of the valves and the extra complication of the mechanism. In 1878 a compound locomotive was tried, but again the results were unsatisfactory. It is to be remarked that compounding involves a more or less complete transformation of the locomotive, and that consequently on account of expense it cannot be applied to all the engines already existing. It appears therefore to be simpler to try to get increased economy by a change which will only involve replacing the cylinders and part of the valve-gear. These considerations led the Company to try the system described below, recommended by Messrs. Durant and Lencauchez.

The immediate aim of the experiments was to prolong the expansion, and to suitably control the compression, while still using a link-motion gear. In the first trial made, on engine No. 67, the admission slide-valve was driven by a Gooch link-motion, and had certain peculiarities of construction, the object being to double the area of opening and to partially relieve the pressure on the face. The exhaust slide-valve, placed at the lower side of the cylinder, received its motion from the cross-head. Exhaust began when the piston had still 22 per cent. of its stroke to perform, and compression began when it had done 22 per cent. of the return stroke. At low speeds the results were satisfactory, but at high speeds the great amount of compression was objectionable.

In the second trial, another locomotive, No. 76, had two exhaust-valves of the Corliss type driven from the cross-head, and the cylinders were larger. Here again the compression was too great for high speeds.

In the next arrangement tried, the rods driving the admission- and exhaust-valves worked in the same link, and were connected to each other, so that they were displaced simultaneously; the same engines on which the above experiments were made being used, cylinders and valves being unaltered. In engine No. 67 release took place when 25 per cent. of the piston-stroke remained to be done, and compression began when 30 per cent. of the return stroke had been done. The results were entirely satisfactory, reversing being easily performed, and a greater amount of work for the same weight of steam being obtained than with ordinary locomotives.

As a result of the preceding experiments, new cylinders with four Corliss valves and the smallest possible clearance spaces were fitted to engine No. 67, and this design has been applied to all new passenger-engines for the last year and a half. The advantages of this arrangement are as follows:—

1. The conditions relating to cooling of the steam on entering the cylinder are better than in the ordinary engines, since the sides of the admission-ports are not cooled by the escape of the exhaust-steam.

2. The fall of pressure is less, since the port area is nearly double.

3. The diminution of the clearance volume, prolongation of the expansion, lessening of the compression, are all favourable to the more efficient use of the steam in the cylinder.

4. The back-pressure is lessened by the diminution of the clearance volume, since the quantity of exhaust-steam to be got rid of is reduced.

5. The friction of the four Corliss valves is less than that of an ordinary slide-valve.

6. The arrangement of the exhaust-valves at the lower side of the cylinder allows a natural drainage to take place, and drain-cocks are therefore unnecessary.

The design has been applied to eight express engines, and is being applied to three express and three goods engines in course of construction. One of these engines coming into the shops for overhaul after running 40,200 miles, the valves and valve-gear were examined. The wear of the valves and the joints of the link-motion was practically nil. With ordinary locomotives the valves and faces usually require adjustment after running about 15,000 miles. The consumption of fuel is 15·2 per cent. less than the average in the ordinary engines doing the same work. The expenditure of lubricant is a little higher, being 51 as against 42 grams per kilometre.

The Paper is accompanied by three plates of drawings and a plate giving an external view of an engine fitted with the new gear.

A. S.

Experiments with Serve Tubes made on the Northern Railway of France. By — Kéromnès.

(Mémoires et Compte rendu de la Société des Ingénieurs-civils, July 1893, p. 42.)

The Serve tubes are now being largely introduced in marine boilers, and have been for some time in use on locomotives of the Paris, Lyons and Marseilles Railway in France.[1] The results of their trial on that railway were so satisfactory that the Northern

[1] See "Revue Générale des Chemins de fer," April 1893, p. 178.

Railway adopted them for three locomotives whose tube plates required renewal, and for fifteen new locomotives under construction at Belfort, and the results obtained were excellent. The Serve tube differs from ordinary tubes in having eight internal radial ribs which serve to increase the heating surface.

The Northern Railway, in their first trials of these tubes, followed the example of the Paris, Lyons and Marseilles Company in using tubes of 2¾ inches diameter instead of the smooth tubes of 2 inches previously in use. The number of tubes was greatly reduced, in some cases by one-half, while the heating surface was at the same time largely increased, and the total section of the tubes open to the passage of the gases somewhat increased also. The surface in contact with the water was of course reduced, a matter of no consequence, since it is in any case greater than is necessary to transmit the heat at the rate at which it is taken up from the gases.

These trials being so satisfactory, it became a question whether Serve tubes of 2 inches diameter could not also be made to serve, since, if not, they could not be introduced except where new tube plates were wanted. The experiments to which this Paper is devoted were therefore undertaken. They were made with a stationary boiler of locomotive type, forming one of a group of four at the railway workshops of Forges de la Chapelle. Two fans, driven by Brotherhood engines, were used to represent the effect of the blast. The grate had an area of 14·4 square feet; the tubes were one hundred and sixty-six in number, 1·97 inch in diameter, and 14·65 feet in length. The heating surface in the furnace was 82 square feet, and in the tubes (before alteration) 1,151 square feet; the steam pressure was 92·4 lbs. per square inch. The boiler under trial was tested alone, and also when working along with the other boilers.

The first Serve tubes tried had ribs of a depth of 0·43 inch, which raised the heating surface from 1,233 square feet to 2,336 square feet, but reduced the section open to the passage of the gases from 2·84 square feet to 2·56 square feet. With natural draught the vaporization per lb. of coal was very slightly increased, the same amount of steam being made in the day with a somewhat reduced consumption. With forced draught, however, the production of steam per lb. of coal was increased by as much as ⅛th, the best effect being got with a vacuum lying somewhere between 1¼ and 2¼ inches of water. The temperature in the smoke-box was lowered from 518° Fahrenheit (with smooth tubes), to 338°. The temperature of the steam being 322°, it is seen that very little more use could possibly have been made of the combustion. The objection, however, to this experiment was that the tubes got often choked with ashes; and therefore a tube of 2·05 inches diameter was tried with ribs 0·35 inch in depth, the ribs extending to a length of 8·2 feet only, and the rest of the tube being smooth. The section for the passage of gases was now increased to 2·69 square feet, and less choking of the tubes

occurred; the efficiency was much the same as in the preceding case. Tubes of the same size were therefore tried with ribs of 0·275 inch projection, both extending the whole length of the tube, and also for a length of 8·2 feet only. The results were much the same whether the rib was of the full length of the tube or not, and showed a vaporization 25 per cent. in excess of that of smooth tubes for the same amount of fuel, with a draught of 3 inches water-pressure.

With tubes of this depth all difficulty as to choking disappeared. With the shallower ribs it was found that a greater draught gave maximum economy than with deeper ribs. The tubes used in these trials have now been fitted on two locomotives under repair, with an increase of weight in one case of 154 lbs., and in the other of 220 lbs. Fresh experiments will be made with them in service.

C. F. F.

Improvement of the Territet-Glion Wire-Rope Railway.

By C. Chessex.

(Bulletin de la Société Vaudoise des Ingénieurs et des Architectes, 1893, p. 93.)

This railway, situated at the eastern end of the Lake of Geneva, ascends a steep hill known as the Rigi Vaudois. It commences at the lake shore, and when first opened for traffic in 1883, terminated at the village of Glion, 1,000 feet above the lake.[1] The motive-power is water, it being a balance incline with tank-fitted cars, full or empty, according to the direction of travel.

The extension of the line to "rochers de Naye" and the general increase in traffic has led to the carrying out the improvements described in the Paper.

Formerly the lower portion of the incline was on a grade of 1 in 3·33 (30 per cent.), which, at its junction with the main incline of 1 in 1·75 (57 per cent.) was on a curve of 68 chains radius. In 1891 this was altered by making the lower portion level for a length of 754 feet from the lower station, and laying it out on a parabolic curve, to the junction with the old incline of 1 in 1·75, which extends for a distance, measured horizontally, of 1,097 feet and terminates at the upper station. The parabolic curve adopted was such as to limit the maximum strain upon the rope to 1,500 times its weight per lineal metre.

This modification has given satisfactory results both as regards the consumption of water and the better regulation of the speed, as, since the opening of the Glion-Naye Extension, the altered conditions have allowed of the adoption of cars with forty places instead of the old ones holding thirty only, and this with scarcely

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[1] Minutes of Proceedings Inst. C.E., vol. xcvi. p. 406.

any increase in the diameter of the cable, which is now $1\frac{3}{8}$ inch (35 millimetres), and weighs 8·47 lbs. per yard.

Also, whereas formerly the quantity of water required in the descending car was 6·80 tons, and that sometimes scarcely sufficient, the large cars now in use only require 5 tons.

Although the system of brakes adopted in the first instance had proved satisfactory, including the automatic brake, which, however, had never been called upon to act spontaneously, it was decided to fit up the new cars with brakes better adapted to the maintenance of regularity of rate of speed. The new car when empty weighs 8·66 tons (8,800 kilograms) and has seat-room for forty passengers and standing room for ten more; the latter space, however, is usually devoted to baggage. The car, with its tank filled, but without passengers, weighs about 13·68 tons, as against about 11·81 tons (12,000 kilograms) for the laden car with its tank empty.

The maximum speed is 2·68 miles per hour, the whole journey occupying nine minutes; the new cars have been in use since April, 1893.

Each car is provided with the following brake-power, namely, two brakes either of which is sufficiently powerful to arrest the car when completely laden, even in the case of a break of cable; a speed regulator, and an automatic clutch in case of the cable failing. Of the two first mentioned brakes one is automatic and the other manual.

In the Paper the description is aided by a reference to a large-scale diagram of the under-frame of the car, showing the brake arrangements and centrifugal speed-regulator.[1]

D. G.

Trial of a Steam Snow-Shovel. By — Bergk.

(Der Civilingenieur, 1893, p. 481.)

The snow-shovel was built by the Görlitzer Maschinenbau-Aktiengesellschaft, and cost £2,250. The machine has the external appearance of a covered goods wagon, with three axles and side windows, and weighs 40 tons. A twin steam-engine, of 600 indicated HP., making 100–120 revolutions per minute, is placed inside the wagon. The shovels project in front of the wagon. Three locomotives of the heaviest type are coupled to the machine, the one nearest being used to supply steam to the engine.

A bank of snow had been prepared on January 10th, 1893, on the railway near Görlitz, to test the machine. The bank was 80–100 metres long, 1·5–2·5 metres high, and $3\frac{1}{2}$ metres wide,

[1] In this Paper no reference is made to the centre rack-rail described in the earlier abstract referred to in the foot-note.—D.G.

and had been allowed several days to harden, so that it required considerable force to be applied with an ordinary shovel to make any impression on it.

The three locomotives exerted a horizontal pressure of 12 tons. The bank was cut through by the rotating snow-shovels in seven-and-a-half to eight minutes, the snow being thrown aside an average distance of 30 metres.

A. S.

A Blasting-Cartridge for Mines. By J. W. Haughee.

(Journal of the Illinois Mining Institute, 1893, p. 108.)

The Author calls attention to a water-proof and air-tight cartridge capable of standing an hour in water without becoming affected. This cartridge, which is manufactured by the Peters Cartridge Company, of Cincinnati, Ohio, consists of a thoroughly water-proof paper tube filled with blasting powder of any sized grain desired, and having the ends closed and sealed by tightly-fitting firmly-secured water-proof disks. The cartridge is manufactured in two sizes, the larger being 2 inches and the smaller 1½ inch in diameter, the length varying from 4 to 36 inches. One of the disks closing the ends has an aperture at one side of the diameter required for the insertion of a blasting-barrel, this aperture being temporarily closed by means of a plug punched from the disk and re-inserted. When the cartridge is to be used the plug is punched out and downward into the powder, the needle or blasting-barrel is inserted in the aperture, and the cartridge is placed in position, tamped, and fired as usual. The blasting-barrel used with this cartridge has a thread cut on the end to be inserted, and is screwed into the aperture of the disk so as to make a water-tight joint. The water-proof paper tube or shell enclosing the powder consists of two thicknesses of brown paper treated with heated oil and resin. The cartridge is, according to the Author, used in many American mines with great success where wet holes have to be fired.

A. H. C.

Recent Observations on Electricity in Coal-Mining.

By Elmer A. Sperry.

(Journal of the Illinois Mining Institute, 1893, p. 103.)

In this Paper the Author briefly summarises the advances that have been made in the United States during the present year, some under his personal supervision or observation, in the application of electricity to coal-mining.

Improvements in electric haulage have been largely in the direction of giving increased flexibility to the wheel-base, and rendering the same more complete. To obtain perfect adhesion on a mine track, which, from many causes, becomes very uneven, the wheel-base should be entirely flexible, dividing up the total weight of the locomotive so as to distribute an equal amount upon each driver contact. This was at first only partially accomplished by a spring over the journal housing from a rigid frame. It has latterly been found that a system of links and rockers somewhat similar to those used in the latest construction of heavy freight locomotives in America is the best arrangement for electric haulage cars in mines. The mechanically coupled driver has been found to be far superior in tractive effect, the tendency of one driver to slip being checked by all the others instead of by its fellow only, as is the case where separately driven axles are employed. For average conditions the locomotive is, in the Author's opinion, the best system of haulage. Where heavy grades in mines exist against the loads, say, from 5 per cent. upwards, and in many other instances, an endless or a tail-rope haulage may be found preferable to electric haulage; but either of these older systems may in most cases be electrically driven with great saving over compressed air or steam.

The Author gives the following rule for determining the proper rail section where electric locomotives are used in coal mining: Divide the weight of the locomotive resting on the drivers in tons by the number of drivers; for dry entries, multiply the number so obtained by 12½, and for wet entries by 15; this gives the number of pounds per yard of rail that should be employed for the best effect.

Among the latest improvements in electric long-wall coal cutters is a specially low type for thin seams, involving the use of a much more powerful motor than has heretofore been applied to this class of machinery, the capacity for power having been increased 75 per cent. This has necessitated a heavier and stronger cutting mechanism, and steel gearing throughout. Trouble from moisture in and about the electrical part of the machine has almost totally disappeared. The method of holding the knives or cutters has been improved, necessitating a new form of cutter. The width of the machine has been kept under 24 inches, allowing the props to be used in close proximity to the face, and rollers have been provided upon the machine for passing the rails from the rear to the front of it, after the machine has travelled along the rails. Other improvements are noted, among which is the new removable truck which has been devised for running the machine upon the mine track when changing it to the point of starting in with the cut.

Very little advance has been made in the past year in breast machines, which have many weak points.

The pick machine has recently undergone considerable change in detail, strengthening of the main structure, &c. It strikes a

heavier and more effective blow than the same class of machine operated by compressed air; and the Author is of opinion that it will be found to fulfil the requirements and to be of general service in and about a mine, among sulphur balls and in hard ground, and under conditions in which other machines cannot be employed.

Improvements in dynamos during the present year have been in matters of smaller detail, tending to increase the output per given weight of machine and rendering it more durable and stable. A special mine-type of dynamo is preferable to the ordinary type in having fewer delicate parts, and being in general less liable to derangement, and designed for specially hard service through any number of hours per day.

A. H. C.

Tail-Rope Haulage. By John L. Dixon.

(Journal of the Illinois Mining Institute, 1893, p. 31.)

The Author maintains from his own experience that the tail-rope system of haulage is the most satisfactory of all those now in operation, for the reasons that it is more self-contained, more positive, more easily controlled on up and down grades, and less liable to get out of order.

The tail-rope works well wherever it is placed, and whether the roads be curved, level, or straight. If the plant be properly constructed, it will work side branches with certainty and economy, all that is required being a good single track and an engine fitted with a pair of drums, one drum being intended for the tail-rope, and the other for the main-rope.

The Author describes this well-known and extensively adopted system in detail, giving some useful figures showing the required dimensions and the weight of the various parts of the installation, together with the engine power required in certain circumstances; and a discussion of the Paper follows.

A. H. C.

The Separation of Blende from Pyrites. By W. P. Blake.

(Advance proof. Transactions of the American Institute of Mining Engineers, August 1893.)

In south-west Wisconsin large deposits of zinc ore occur in the form of sheets of blende and pyrites (marcasite), sometimes well separated, but often as a close mixture, which cannot be brought up to the proper standard for reduction by mechanical methods

alone, the specific gravity of the two minerals being so nearly the same that a marketable concentrate cannot be obtained by jigging. Experiments in this direction were made by the Wisconsin Lead and Zinc Company, who erected two mills and turned out hundreds of tons of concentrates of different sizes, which were unsaleable owing to the high percentage of pyrites retained. Galena could be easily separated from such mixed ore, but blende and pyrites were inseparable, and neither was obtained in a marketable form, as the smelters of zinc ores in the Mississippi valley will not take those containing more than 7 per cent. of iron. The method devised by the Author is similar to that adopted in tin dressing, the metallic minerals being first separated from the bulk of the earthy waste as a mixed concentrate, which is subjected to an oxidizing heat in a furnace described in a former Paper,[1] which entirely or partially decomposes the pyrites, while blende and galena are scarcely, if at all, changed, the former mineral scarcely losing its brilliant lustre, while the latter is hardly tarnished. Beyond a slight breaking-up of the fragments of blende by decrepitation, there is no change of form in the concentrate except as to the pyrites, which suffers a change of bulk with the loss of its sulphur.[2] The pieces of marcasite swell, expand, crack, and exfoliate, becoming more bulky than before roasting; and the blende fragments, being reduced in size by decrepitation, a condition favourable to separation by jigging is obtained. The products of the latter operation are clean blende, or "dressed jack," and tailings, consisting chiefly of red oxide of iron, which serve as the basis of an excellent metallic paint, their value being enhanced by containing a small proportion of oxide of zinc. Any lead ore present is removed in the first compartment of the jigging machine, the furnace operation having split off the blende, leaving the galena free. This is another important advantage, as blende containing more than 2 per cent. of lead is unfit for the production of the best spelter. From a first raw concentrate, containing blende 25 per cent., pyrites 25 per cent., and galena 5 to 10 per cent., and the remainder dolomite and flint, after roasting and second jigging, marketable blende is obtained of a very good quality, assaying in the best samples over 62 per cent. of zinc, less than 3 per cent. of iron, and 1 per cent., or in some cases below ½ per cent., of lead.

Complete and even roasting is essential for the success of the process, as every particle of bisulphide of iron must be decomposed. Even a remnant or kernel of unchanged pyrites will cause the fragment to remain with the blende. The outer

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[1] Transactions, xxi. p. 943.

[2] This is essentially a Brunton calciner, in which the revolving bed, instead of being a smooth cone, is stepped, forming a series of annular shelves 18 inches broad, with a maximum diameter of 16 feet at the bottom. The ore is fed through a funnel in the apex of the roof, and is discharged at the circumference of the bottom shelf.

oxidized coating may be broken off and pass into the tailings, but the unchanged part will not go over, and such imperfectly roasted ore will have to go back to the furnace. It has been found that by proper manipulation the pyrites, when reduced to the magnetic form by the removal of the second atom of sulphur, may be left in a condition to be attracted and removed by the magnet. Experiments have been made upon this subject by Mr. J. W. Meier, who finds that mixed ores, with 33 to 36 per cent. of zinc and above 10 per cent. of iron, may, by roasting in a muffle and magnetic separation, be easily converted into concentrates with 50 to 54 per cent. of zinc and only about 4 per cent. of iron. The chief difficulty is the conversion of the whole of the pyrites to the magnetic state, as fragments fully converted into ferric oxide cannot be removed by the magnet, and a partial reduction of these from ferric to magnetic oxide by the action of solid or gaseous carbon seems to be necessary after roasting. This method might probably be applied with advantage to the ferruginous calamines of Wisconsin, it having been proved to be perfectly successful with similar ores at Monteponi[1] in Sardinia.

The method of dressing with intermediate roasting described by the Author has been in operation for nearly two years at the Helena Mill near Shullsburg, in Wisconsin, where hundreds of tons of high class zinc blende have been produced and sold to the smelters from ores that could not be cleaned by the method of hand-picking alone. Much of the waste of the dressing-floors accumulated during many years of working has thus been utilized.

H. B.

On the Determination of Carbon in Iron. By A. Ledebur.

(Verhandlungen des Vereins für Beforderung des Gewerbfleisses, 1893, p. 280.)

This memoir contains the results of an investigation into the accuracy of the methods in general use for the determination of carbon in the different forms of cast-iron and steel. The materials examined were:

1. A mottled charcoal pig-iron, containing 0·79 silicon and 1·54 manganese.
2. A white pig, with manganese 3·0 and phosphorus 2 per cent.
3. A crucible tool steel, with manganese 0·11 and 0·12 silicon.
4. A basic mild steel recarburized by Darby's process, with 0·54 manganese.
5. A similar steel, with 0·56 manganese. These two samples were distinguished as Darby steel A and B.

The samples for analysis, with the exception of the white pig-

[1] Minutes of Proceedings Inst. C.E., vol. cix. p. 483.

iron which was crushed in a steel mortar, were prepared in the form of drillings by a blunt drill free from grease. This form was selected as being best suited to the ordinary conditions of iron works practice, while it allowed the ready detection and separation of foreign matters. It was also considered that methods giving good results with borings would be equally suitable with the metal in filings, while the reverse condition might not always be true.

The processes examined were the following :

A. Determination of total Carbon.

1. *Direct combustion with Oxygen.*—Three experiments upon grey pig-iron, continued for 3½ hours in the first instance and 6¼ hours in the second, and gave 3·254, 3·515, and 2·872 per cent. of total carbon respectively. With longer-continued heating and the sample in filings, instead of borings, probably more concordant results might have been obtained, but having regard to the ease with which mistakes may be made, the bulky character of the apparatus, and the slowness of the operations, this method seems to be unsuited for practical use.

2. *Oxidation by Chromic and Sulphuric Acids.*—In this, known as Jüptner and Gmelin's method, the metal is dissolved with simultaneous oxidation of the carbon to carbon dioxide by boiling it with a saturated solution of chromic acid mixed with about twenty times its volume of strong sulphuric acid for periods varying from 1 to 2 hours. Four experiments were made with grey pig-iron and two with tool steel, but the results were in all cases too low. This is due to the circumstance that there is invariably some hydro-carbon gas formed during the solution which escapes oxidation.

3. *Oxidation by Chromic and Sulphuric Acids with combustion of Hydro-Carbons.*—In this modification of the preceding method, which is due to Sarnstrom, the gases evolved from the dissolving vessel are passed through a heated tube containing cupric oxide, or a red-hot capillary tube of platinum, in order to decompose any hydro-carbons that may be formed. This, being one of the best methods in regard to accuracy, was subjected to numerous trials with the apparatus modified in several different ways to meet particular causes of error. The results obtained with all the samples were about the same with either kind of supplemental combustion; but the Author considers the platinum tube to be preferable to that with cupric oxide, as the latter requires frequent renewal and takes a longer time to heat, fifteen to twenty instead of one or two minutes. The time required for the complete analysis is from ¾ to 1 hour for the preliminary heating, followed by 2 to 2½ hours' continuous boiling, a rapid current of dry air being maintained through the apparatus without interruption. When the temperature of the air in the laboratory exceeded 20° C. (68° Fahrenheit) the proportion of carbon found appeared to diminish as the temperature rose. This was traced to the circumstance that the dried air used in driving

the carbon dioxide out of the combustion-apparatus into the potash bulbs, took up moisture from the potash solution which was not reabsorbed by the calcium-chloride tube at the tail of the apparatus, and it was only by the addition of a supplemental tube with strong sulphuric acid that this source of error was avoided.

4. *Removal of Iron by Cupric Sulphate.*—This method was tried with cast-iron, the action of blue vitriol solution being continued from 4½ to 7 hours, the separated carbon being subsequently oxidized by chromic and sulphuric acids. The results were too low, owing to loss of carbon as hydro-carbon gases, and therefore the experiments were not continued.

5. *Removal of Iron by Cupric Sulphate and combustion of Hydro-Carbon.*—This is a modification by Sarnstrom of the preceding method, an accessory combustion apparatus either by cupric oxide or heated platinum being used in method No. 3. The results were as a rule correct, though in some cases rather high, leading to the supposition that sulphur dioxide may have been formed by the action of the strong sulphuric acid upon metallic copper, separated during the solution of the iron, but a long series of experiments with a view to determine this point led to no result.

6. *Removal of Iron by Cuprammonium Chloride.*—This method, introduced by McCreath, was much used about ten years since, but the discovery of several sources of error, real or supposed, have led to its abandonment. When tried with cast-iron it gave good results, but with steel was less satisfactory. In some cases the figures obtained were too high. This may be due to the sal-ammoniac, which, when made from ammonia derived from tar distilling, is apt to contain carbon. In any case the results depend so much upon manipulation that the method cannot be recommended for general use.

7. *Removal of Iron by dry Chlorine.*—In this method the sample contained in a porcelain boat is heated in a current of dried chlorine gas until the whole of the iron is volatilized as ferric chloride, and the separate carbon remaining in the boat is subsequently oxidized by chromic and sulphuric acids. This, if properly conducted, is probably the most accurate of all the methods, but the Author considers that the results may be too high if the carbon residue is burnt without being previously washed to remove traces of chloride remaining. And in the latter operation losses may easily occur, owing to the escape of finely divided carbon through the filter. For this reason the results obtained with grey cast-iron are better than with white iron or steel, as in the latter the carbon, being separated in an extremely light form, is more readily carried away in the gaseous current, or by washing than in the former, where it is mostly present in the denser form of graphite.

The general results of the investigation are contained in the following Table, which gives the average figures obtained by each method, and their extreme variations :—

—	Carbon per cent.				
	Cast-Iron.		Crucible Steel.	Darby Steel.	
	Grey.	White.		A.	B.
1. Combustion in oxygen—					
Average result . . .	3·214				
Extreme variation . .	0·643				
2. Solution in chromic sulphuric acid alone—					
Average result . . .	3·565	..	0·732		
Extreme variation . .	0·418	..	0·045		
3. Solution in chromic sulphuric acid and accessory combustion—					
a. With cupric oxide—					
Average result . . .	3·955	3·225	0·870	0·555	0·391
Extreme variation . .	0·024	0·064	0·059	0·009	
b. With platinum tube—					
Average result . . .	3·871	3·248	0·859	0·550	0·380
Extreme variation . .	0·209	0·028	0·054	0·016	0·046
4. Solution of iron with cupric sulphate only—					
Average result . . .	3·636				
Extreme variation . .	0·237				
5. Solution with cupric sulphate and accessory combustion—					
a. With cupric oxide—					
Average result . . .	3·747	..	..	0·646	
Extreme variation . .	0·132	..	..	0·033	
b. With platinum tube—					
Average result . . .	3·760	3·387	0·903	0·664	
Extreme variation . .	0·043	0·046	0·009	0·031	
6. Solution with cuprammonium chloride—					
Average result . . .	3·968	..	0·927	..	0·404
Extreme variation . .	0·236	..	..	..	0·026
7. Removal of iron by chlorine—					
Average result . . .	3·999	3·282	0·869	0·593	0·368
Extreme variation . .	0·112	0·018	0·089		

B. Determination of graphite.

The grey pig-iron of the previous experiments was examined by eight different methods for the determination of its contained graphite. In five of these, hydrochloric acid was used as the solvent at temperatures varying from 50° Centigrade to the boiling point for periods ranging between one and twenty-four hours, the

residue being washed with water, potash lye, alcohol and ether, and dried at 100° to 120° Centigrade. In the remaining three, nitric acid was used, the residue being washed with water alone, but dried at different temperatures as before. The results obtained were too high when hydrochloric acid was used at a low temperature, as then the carbide, or semi-combined carbon, is not decomposed, and is ultimately burned with the graphite. This error is avoided by the use of nitric acid, which has the further advantage of dispensing with the troublesome washing with alcohol and ether. In either case it is necessary to dry the residue at a higher temperature than 100° Centigrade, or there may be a considerable error in excess. The final combustion may be effected equally well with oxygen or chromic-sulphuric acid; the mean result obtained by the former was 2·745, and by the latter 2·749 per cent.

A second portion of the memoir contains an investigation of the Eggertz process as applied to steel of different kinds, both as cast and when forged and rolled. This confirms generally the accuracy of the process and its applicability to practical uses. One sample of Bessemer steel behaved in a very anomalous manner, giving very irregular results both when examined alone and when used as a standard. This the Author considers may have been caused by its being water-cooled before use, but the fact could not be established.

In his concluding remarks the Author considers that the two Sarnstrom methods, No. 3 and 5, are to be recommended for general use; the former being best adapted for graphitic iron, and the second for other forms of iron and steel with combined carbon, which are not easily decomposed, except the iron be removed by copper sulphate. For check purposes the chlorine method is of great value.

The volumetric methods of determining carbon by measuring the carbon dioxide produced, instead of weighing it, have not up to the present time proved sufficiently accurate for practical use, although it is not improbable that they may shortly be improved so as to be equal to the older methods.

H. B.

An Examination into the accuracy of Methods used for Determining Carbon in Iron. By Dr. Göttig.

(Verhandlungen des Vereins zur Beförderung des Gewerbfleisses, 1893, p. 321.)

This is a long memoir by the professor of chemistry at the Royal Artillery and Engineer School, Berlin, recording the results of a systematic investigation into the methods in use for determining carbon in iron. It is generally similar in character to that of Professor Ledebur, noticed in the preceding abstract, but a larger number of processes, nearly seventy, seem to have been tested. For the details of these, reference must be made to the

original. The practical conclusions deduced by the Author are as follows:—

In graphitic iron the total carbon may be rapidly and accurately determined by direct oxidation with chromic and sulphuric acids, if the weight of chromic acid used is from twelve to fifteen times that of the iron.

When the sample is coarse grained a modification of Weyl's method, using a basket of fine platinum gauze in connection with the positive electrode, is recommended to keep the separated carbon together.

The methods of combustion used in organic analysis and that of removal of the iron by heating in chlorine are inadmissible, owing to the extreme difficulty of effecting the perfect combustion of graphite.

In non-graphitic iron, Wöhler's method of separation by heating in chlorine and subsequent combustion of the residue in oxygen is to be recommended when a long series of determination has to be made, as from eight to ten samples may be treated with chlorine at one time. It is, however, unsuited for highly manganiferous pig metal, as manganese chloride, unlike that of iron, cannot be completely volatilized, and must be washed out of the carbonaceous residue before combustion, which operation is likely to be attended with the loss of some of the finely-divided carbon.

When only a single determination is to be made direct oxidation with chromic and sulphuric acid, with a supplementary oxidizing tube for hydrocarbons, is to be recommended.

When a copper salt is used to separate the carbon before oxidation the reduced copper need not be removed, except when the residue is to be burnt with oxygen, as in that case it is difficult to obtain complete oxidation of the mixture of copper and carbon.

Weyl's method with the platinum basket, described above, may be conveniently used when the metal cannot be powdered.

In the determination of graphite, nitric or hydrochloric acid may be used as a solvent. The former is preferable, as being most rapid, but the results may be a little low, owing to a small oxidation of graphite during the process. Hydrochloric acid gives good results if the assay is boiled for several hours, in which case the washing out of the residue with water, potash lye, alcohol, and ether is unnecessary.

H. B.

On Sulphur in Cast-iron. By W. J. Keep.

(Advance proof. Transactions of the American Institute of Mining Engineers, August 1893.)

The Author has for six years been trying to verify the received belief that sulphur is in every way injurious to cast-iron, and has made numerous experiments with artificially sulphurized cast-iron up to 2 per cent. of sulphur, both grey and white, the results of which are recorded in the Paper. The conclusion finally reached is that the proportion of sulphur retained by grey cast-iron cannot materially injure the iron except by increase in shrinkage, which in the extreme ends seems to be from 0·168 to 0·194 inch per foot. The general testimony is that most of the sulphur present in pig-iron is lost in re-melting, and that it is impossible that it can be re-absorbed to any damaging extent from the fuel. The influence of sulphur is diminished by increase of carbon or silicon. In wrought-iron, which is practically free from these elements, a small amount of sulphur is said to do great harm, and such iron will take up sulphur in considerable quantity. In steel containing less than 1 per cent. of carbon and practically no silicon a few hundredths per cent. of sulphur cause decided injury. With carbon increases to saturation as in white cast-iron, not more than 0·50 per cent. of sulphur can be retained, and as silicon increases, producing graphitic carbon, it is still more difficult to find sulphur in the metal, as it will not absorb it from the outside.

The influence of sulphur on all cast-iron is to drive out carbon and silicon, to increase shrinkage, and in general to reduce strength, but in practice sulphur will not enter the iron in the foundry to a sufficient extent to realize these defects. When, as happens sometimes, iron which was grey when put into the cupola comes out white with increased shrinkage and chill, and often with decreased strength, the change is due to loss of silicon and can be remedied by re-melting with an appropriate addition of ferro-silicon or other highly silicized metal.

That highly sulphurized iron cannot be produced in practice is evident from the fact that ores are used (in America) containing over 3 per cent. of sulphur with fuel containing nearly 1 per cent. of the same element; but all smelters are agreed that not more than 0·75 per cent. of sulphur can be found in white iron, 0·25 per cent. in mottled, and not more than a few hundredths in grey foundry iron. The experiments show that 0·50 per cent. of sulphur will not exert any appreciable deleterious influence, and even that may be at once corrected by a slight increase in silicon.

Remembering that the only noticeable effects would be to slightly increase the shrinkage, the above conclusion seems to be proved by the fact that, in a foundry working with a substantially

uniform mixture, returning from 25 to 40 per cent. of its metal to the cupola to be re-melted over and over again each day, the castings will almost invariably have a less shrinkage than the average of the pig-iron used, and yet they will probably contain more sulphur and less silicon than the pig-iron originally did.

The depth of chill does not seem to be influenced by these small proportions of sulphur, neither is there any evidence to show a relation between sulphur contents and strength either in pig-iron or ordinary castings. The Author hopes at a future time to explain why inferior castings are from time to time produced from a uniform mixture of pig-iron.

H. B.

The Working of Charcoal Blast-Furnaces. By J. Hörhager.

(Berg- und Hüttenmännisches Jahrbuch der k.k. Bergakademien, 1893, p. 1.)

The Author gives a detailed study of the results obtained with three charcoal blast-furnaces. The investigations extended over several years. The three furnaces were about the same size, the total height being 40 to 42 feet, and the capacity varying from 1,270 to 3,300 cubic feet. The section of the furnaces varied considerably. The yield was lowest with furnace A of wide section, and greatest with the narrow furnace C. Of the furnaces A and B, working under similar conditions, B with a capacity of 2,100 cubic feet gave the same output as A, with a capacity of 3,300 cubic feet. At the same time the consumption of fuel was less. Furnace C had relatively the highest output, but it had also the highest fuel-consumption. The Author's calculations show that for every kilogram of pig-iron, there are on an average 3,100 to 6,000 calories yielded, of which nearly 90 per cent. is due to combustion in the furnace itself, while 10 per cent. is due to the hot blast. The following Table shows the average distribution of the heat yielded:—

	Per cent.
Reduction of iron, manganese, and silicon	42
Smelting of slag and pig-iron	13
Expulsion of carbonic anhydride and moisture	10
Heat of waste gases	17
Cooling twyers	3
Loss of heat	15
Total	100

The Author's calculations show that two-thirds of the heat of the waste gases is utilized, and one-third is lost. The following summary shows the manner in which this takes place:—

Per Kilogram of Pig-Iron.	Furnace B.				Furnace C.	
	Foundry-Iron.		Forge-Iron.		Forge-Iron.	
	Calories.	Per cent.	Calories.	Per cent.	Calories.	Per cent.
Carbon in escaping gases gives on complete combustion	11,150	100	6,706	100	10,423	100
(a) *Heat utilized*:—						
For the blast-furnace process . . .	4,772	43	2,846	42	4,094	40
For heating the blast	650	6	260	4	528	5
For steam-raising .	2,130	19	1,064	16	1,092	10
For roasting . . .	..	..	..	..	637	6
Total useful heat	7,552	68	4,170	62	6,351	61
(b) *Heat lost*:—						
In heating the blast	1,817	16	1,175	18	767	7
In steam-raising .	1,571	14	1,089	16	943	9
In roasting . . .	..	..	..	..	658	6
By excess of gas .	..	..	..	..	1,543	15
Other losses . . .	210	2	272	4	161	2
Total heat lost .	3,598	32	2,536	38	4,072	39

B. H. B.

Experiments on the behaviour of Phosphoric Acid in the Blast-Furnace, and the influence of Phosphorus on the composition of Pig-Iron. By N. Kjellberg.

(Jernkontorets Annaler, 1892, p. 180.)

Until recently it has been generally supposed that the whole of the phosphorus present in a blast-furnace charge passed into the iron, and that no appreciable quantity was to be found in the slag. There must, however, be a point at which the iron becomes saturated, and the phosphorus must then combine with the slag. With basic slags, and at low temperatures, much of the phosphoric acid passes into the slag; but it is not easy to determine in advance how much of this impurity will be ultimately found in the slag, and how much in the iron.

In Sweden there are large deposits of iron ore rich in phosphoric acid, especially at Grängesberg and Gellivara, and the Author's experiments were undertaken with the view of finding the best method of utilizing these ores.

There are three main causes which determine the proportion of phosphorus in slag or metal, namely, the amount of phosphorus

in the charge, the temperature in the blast furnace, and the basicity or acidity of the charge. The experiments were made in a small blast furnace with quantities of 4 tons of ore in each case. The ores were smelted without previous calcination, and their percentage of phosphorus varied from 0·2 to 3·6. Each sample of ore was run with basic and acid slags, and the difference in the composition of metal and slag determined in each instance. The results of the tests and analyses are given in detail in tabular form, and the Author deduces from them the following conclusions:

(1) When the phosphorus in the ore does not exceed 1¼ per cent. neither the low temperature in the blast furnace nor the higher or lower proportion of silica in the slag has any appreciable influence on the reduction of the phosphoric acid. The greater part of the phosphorus of the ore combines with the iron, so that ultimately 90 to 95 per cent. is found in the pig-iron and 5 to 10 per cent. in the slag.

(2) When there is more than 1¼ per cent. of phosphorus in the ore there appears to be a more definite division of the phosphorus between slag and iron. Up to a percentage of 3·5 the greater proportion of the phosphorus unites with the iron; but with a basic charge from 40 to 50 per cent. may be driven into the slag. With a high temperature and acid slag about 95 per cent. of the phosphorus may be brought into combination with the iron.

(3) There appears to be no volatilization of phosphorus in the blast furnace in the case of ores containing up to 3·6 per cent.

(4) The carbon in pig-iron diminishes as the phosphorus increases, especially when the latter is above 3 per cent.

(5) The proportion of silicium also decreases as the phosphorus rises, and a pig-iron containing 4 per cent. of phosphorus may not contain more silicium than steel. The specimens of iron containing most phosphorus were so brittle that the ingots could be broken by a light blow from a hammer.

In the case of ores containing 50 per cent. of iron the pig-iron from the blast furnace will contain about double the percentage of phosphorus existing in the ore. For instance, an ore with 0·25 per cent. will yield an iron with 0·5 per cent. When the percentage of iron in the ore amounts to about 60, it is possible to produce an iron with a smaller proportion of phosphorus when the basic process is employed. For foundry pig-iron, with an amount of phosphorus not exceeding 1 per cent., 60 per cent. ores should be chosen with not more than 0·6 per cent. phosphorus. For pig-iron to be treated by the Thomas basic process, and containing at least 2 per cent. phosphorus, the percentage of that impurity in the ore should not be less than 1·6.

W. F. R.

A New Method of Reducing Metallic Oxides.

By W. H. GREENE and W. H. WAHL.

(Journal of the Franklin Institute, vol. cxxxv. 1893, p. 453.)

Among the reducing agents commercially available for the producing of manganese from its oxides in a state free from carbon besides aluminium, which has already been applied by the Authors, silicon is obviously the most likely one, as the heat developed by its oxidization to SiO_2 is sufficiently in excess of that required to reduce $2MnO$ to permit the supposition that the desired reaction would take place. As, however, pure silicon can only be obtained by complicated and costly means the direct use of this re-agent is out of the question, but it is different with metallic silicides such as ferro-silicon and silicon spiegel, which contain large proportions (5 to 30 per cent.) of silicon with comparatively little carbon, as the latter diminishes as the former increases. These silicides the Authors propose to employ in the production of ferro-manganese and manganese steel, which latter substance under the present condition of manufacture, where ordinary ferro-manganese is added to fluid iron, is limited in its manganese contents by the carbon, 5 to 6 per cent., which is present in the ferro-manganese, and which cannot be eliminated from the steel on account of the affinity of the manganese for carbon.

The chemistry of the operation is exceedingly simple. When ferro-silicon is heated in a crucible or furnace lined with lime or magnesia in contact with a metallic oxide or a mixture of such oxides with lime magnesia and alumina for fluxing, the silicon is oxidized by the oxides, and combines with the fluxes to form a slag, while the metals set free combine to form an alloy. Assuming the oxide to be that of manganese as MnO or Mn_3O_4, and the silicides to contain—iron 88, silicon 10, and carbon 1 per cent., the proportions required to satisfy the equation—$2MnO + Si = SiO_2 + 2Mn$—will be 142 of manganese to every 28 parts of silicon in the silicide, and sufficient earthy oxides to form a properly fusible slag with the silica produced. A further amount of manganese oxide will, however, be necessary, as a certain quantity is invariably taken up by the slag, and so escapes reduction. An experimental charge, containing :—

	Lbs.
Ferro-silicon (Si 10, C 1 per cent.)	100
Manganous oxide	70
Lime	50

Yielded 128 lbs. of an alloy, containing—

	Lbs.
Iron	70·0
Manganese	29·0
Carbon	0·7
Silicon	trace

Assuming that the permissible carbon in a manganese steel is 0·5 per cent., 100 pounds of such an alloy when added to 40 pounds of decarburized iron will give 140 pounds of manganese steel of the composition—iron 78·8, manganese 20·7, carbon 0·5, silicon trace.

In the preparation of manganese steel of the same carbon temper with ordinary ferro-manganese (80 manganese to 5·5 carbon) the resulting metal would contain 92·3 iron and only 7·27 manganese. On the basis of equivalent contents of manganese, 20·7 per cent. of manganese would bring in 1·4 per cent. of carbon, using ordinary ferro-manganese, while by the Author's method, 7·7 per cent. manganese would correspond to only 0·18 per cent. of carbon. By using the oxides of other metals with ferro-silicon, iron alloys with more than 50 per cent. of nickel, with nearly 20 per cent. of chromium, and more than 50 per cent. of tungsten have been obtained, all being relatively low in carbon. Titanium has so far only given doubtful results. Alloys of copper with difficultly reducible metals may be similarly obtained by fusing their oxides with copper silicide.

H. B.

Contributions to the Technology of Alkaline Bichromates.

By C. Haussermann.

(Dingler's Polytechnische Journal, vol. cclxxxviii. 1893, pp. 93 *et seq.*)

The manufacture of alkaline bichromates, which originated in Scotland, was introduced into Germany about 1880, for utilizing chromium oxide obtained in alizarine works, and has subsequently been extended to the treatment of chromic iron ore. This mineral, like that used in England, is principally obtained from Asia Minor, other sources of supply being of minor importance. An average sample from Smyrna was of the following composition:—

Cr_2O_3.	Al_2O_3.	Fe_2O_3.	FeO.	MgO.	CaO.	SiO_2.	CO_2.	Loss.
51·2	12·8	1·25	13·32	12·55	3·15	4·95	0·20	0·38

Roasting.—In the first operation the ore is decomposed by subjecting it to a slow roasting heat in intimate mixture with lime and soda. For this purpose it is necessary to reduce it to the finest possible powder, which can be advantageously done in a No. 4 Gruson ball mill. This is capable of grinding 9 cwts. of chromic iron ore per hour to pass through a 130-mesh sieve without notable residue. The lime and sodium carbonate are equally finely ground, for which purpose the Excelsior mill, of the same makers as used for grinding lime in sugar factories, is recommended by the Author. The essential chemical change produced in the above mixture when heated in air is as follows:—

$$2FeCr_2O_4 + 4Na_2CO_3 + 7O = Fe_2O_3 + 4Na_2CrO_4 + 4CO_2,$$

the lime being added merely to render the charge infusible and to keep it porous. If sodium carbonate were used alone it would melt at the high temperature developed in roasting, and prevent the complete oxidation of the ore. The mixtures used vary considerably from 4½ of ore, 7 of burnt lime, and 2¼ of sodium carbonate in England to 6 of ore, 3 of chalk, and 3 of calcium soda ash in Russia. The former proportions are the most advantageous as allowing about 90 per cent. of the ore to be decomposed. Even less than the theoretical quantity of alkaline carbonate may be used, as the lime then becomes active and calcium chromate is formed in notable quantity.

For roasting the Author recommends a Bicheroux furnace about 25 feet long and 6 feet broad, having a long flue below the bed for heating the air used in combustion to 300° or 400° Centigrade before admission to the fire-place. The bed is broken into three terraces, the charge of 2½ tons of ore being introduced at the highest level at the flue end and shifted every eight hours until it reaches the fire bridge. The heat ranges between the melting point of aluminium at the flue end up to nearly that of gold at the point of discharge. About 4½ tons of good coal are required to finish the charge in twenty-four hours. The progress of the operation is controlled by determining the amount of chromate formed in a sample when extracted by weak acid. The finished product, which is of a yellowish green colour, weighs about 5 per cent. less than the charge, the difference being due to carbonic acid and moisture driven off, in addition to a small amount of flue dust carried over by the flame.

Of the numerous other methods proposed for decomposing chromic iron ore, none have been successful in practice. Among the latest, that proposed by Mr. P. Kestner may be mentioned. In this the powdered ore, mixed with three times its weight of barium carbonate, is heated to 1,200° or 1,300° for one and a half hour, producing caustic baryta and chromic oxide. The baryta is removed by solution in boiling water, and the insoluble chromic oxide is subsequently converted into chromate by roasting with soda. It seems doubtful, however, whether this transformation can be carried out without loss of baryta, as there is always some barium chromate formed, a substance which is not recognized when the mixture is exhausted with acid, its yellow colour in the liquor being masked by the green of the iron salt. If, however, a hot soda solution is used instead of acid, the formation of sodium chromate becomes apparent from the pure yellow tint of the liquor, and barium carbonate will be found in the insoluble part. It would seem, therefore, that Kestner's process can only be practically applied under very special conditions.

Solution.—The extraction of the chromates from iron oxide and other insoluble matters in the furnaced charge is done with an aqueous solution of sodium carbonate, which at temperatures above the boiling point converts calcium chromate into the corresponding sodium salt. The mass is mixed with about twice its weight of

wash-water from a preceding operation, to which about 5 per cent. of fresh soda has been added, and boiled for two and a half to three hours at 120–130° in a cylindrical iron vessel either horizontal or vertical provided with stirring paddles. The heating is done with live steam, which is also used to blow the contents of the vessel into the filter press where the soluble contents are exhausted by systematic washing. The character of the residue varies with the nature of the charge and other circumstances. In one case where ore and lime were used in the proportion of three to four, the composition was found to be

Na_2O.	CaO.	MgO.	Fe_2O_3.	Al_2O_3.	Cr_2O_3.	CrO_3.	SiO_2.	CO_2.	H_2O.	Insoluble.
0·2	46·5	12·2	7·5	5·4	1·0	1·8	1·4	5·2	16·0	1·2

The insoluble part contained fire-brick from the furnace, unchanged ore, chromium oxide, and chromic acid in forms soluble in acids but not in alkalies. These probably existed as chromic chromate, a substance which is formed either by the partial reduction of chromic acid or the incomplete oxidation of chromic ore. These residues, apart from their free lime being used to neutralize acid waste waters, have not hitherto been usefully employed. Latterly it has been proposed by P. Römer to utilize them in the production of carbonate from sulphate of potassium by means of potassium bichromate, lime and carbonic acid.

Apart from the final weak wash-waters, the solution from the filter press, which contains free sodium hydrate as well as neutral chromate, is boiled down in iron pans to specific gravity 1·5, when it would, if cooled, deposit the salt as $Na_2CrO_4+10H_2O$; but at this state it is converted into bichromate without attempting to recover the free alkali, an operation which might be effected by carbonic acid supposing the price of soda to be sufficiently high.

Souring.—In this operation sulphuric acid of 80 per cent. concentration is added to the press liquors until all the free alkali and one-half of that in combination with chromic acid are converted into sulphate, leaving the remainder as sodium bichromate thus:—

$$2Na_2Cr_2O_4+H_2SO_4=Na_2SO_4+Na_2Cr_2O_7+H_2O.$$

The sulphate separates almost entirely as a difficultly soluble precipitate of the anhydrous salt, while the bichromate remains in solution. Before the acid is added, the solution must be heated to prevent separation of the neutral chromate, which can only be decomposed with difficulty when in the solid state. The operation is performed in steam-jacketed iron pans lined with lead. The acid, which must be as free as possible from arsenious acid, oxides of nitrogen, and other reducing impurities, is added until the liquor contains a little free chromic acid, which must, however, be carefully neutralized with soda lye, otherwise the subsequent concentration could scarcely be done in iron pans on account of the destructive action of chromic acid on that metal.

After decantation of the clear liquid, the sodium sulphate remaining in the pan is passed through a centrifugal extractor, and recrystallized to free it from adhering mother liquor.

Finishing.—The bichromate solution from the centrifugal is boiled down in pans heated by a naked fire, such as are used in making caustic soda. At first some sulphate separates, which is fished out and added to that previously obtained. Subsequently there is some reduction of bichromate by the iron of the pan, so that the last portion of sulphate contains a notable amount of chromium in an insoluble form, which is therefore added to the charge in a subsequent roasting operation. When a density of 1·7, corresponding to 1,650 grams of bichromate per litre, is reached, the liquor is filtered hot, and set aside to crystallize. If the cooling is done rapidly with continuous agitation, the salt separates in small needle-like crystals of an orange colour; but if large vats are used, in which the solution is allowed to cool slowly and quietly, the crystals are large, and resemble those of the potassium salt. The composition $Na_2Cr_2O_7 + 2H_2O$ is, however, the same in either case.

The product as finished for market contains from 98 to 99 per cent. of the pure salt, the difference being made up of sulphate and traces of ferric oxide and alumina. The specific gravity of the large crystals, which has not been previously recorded, is given by the Author as 2·6. They dissolve in water with an absorption of heat.

The composition of sodium bichromate changes by exposure to the air, owing to the absorption of water, while that of the potassium salt, which is not very hygroscopic, remains constant. This disadvantage is, however, compensated by lower price and more ready solubility, so that it is possible that the latter salt will in time be entirely replaced by the former. A third commercial form of the salt is obtained by melting the crystals at 100°, and pouring the liquid mass on to enamelled plates, where it solidifies to a cake containing rather less water and more sodium sulphate than the crystals, which is, therefore, less hygroscopic.

The most convenient method of discriminating between the alkaline bichromates, is afforded by their behaviour with absolute alcohol, in which potassium bichromate is insoluble at or slightly above the ordinary temperature of the air, while the sodium salt gives it a yellow colour immediately, and on standing, brown insoluble flakes of chromic chromate separate slowly, or more rapidly by heating.

Under the most favourable conditions, the yield of bichromate does not exceed 90 per cent. of that theoretically obtainable from the ore. A principal source of waste, in addition to those previously mentioned, is to be found in the large volume of insoluble furnace residues, amounting to about 80 per cent. of the charge, which retain a soluble proportion of the chromate liquors, by capillarity, however carefully the pressing and washing may be conducted. This large production of waste material is the weak

point of the process; until it can be avoided no great improvement in the manufacture seems possible.

Potassium Bichromate.—This salt is made by decomposing sodium bichromate with potassium chloride, both salts being dissolved in water—

$$Na_2Cr_2O_7 + 2KCl = K_2Cr_2O_7 + 2NaCl.$$

The older method of heating the ore with lime and potassium carbonate has now been entirely abandoned, as the latter salt, besides being sensibly volatile at a high temperature, is considerably dearer than potassium chloride.

The strength of the solution must be so proportioned that when mixed the sodium chloride may remain almost entirely dissolved, while the bichromate separates. This is best realized with 1,500 grams of sodium bichromate, and 300 grams of potassium chloride respectively, the latter solution being added to and incorporated by rapid stirring with that of the bichromate. The precipitated salt is separated in a centrifugal machine, and redissolved in order to obtain the large crystals required in commerce. These are best obtained from a solution containing 570 grams per litre when hot, which is allowed to cool as slowly as possible, by the use of well covered crystallizing vats of large capacity. The mother liquors are used for dissolving fresh portions of the rough salt, while those from the decomposing pans, which contain from twenty-two to twenty-four parts of bichromate to one hundred of sodium chloride, are concentrated by evaporation, when there is a separation of common salt containing some chrome while the liquor is hot; but when it is brought to 1·38 specific gravity and cooled, crystals of potassium bichromate are formed, which are recrystallized in the manner previously described.

The small quantity of chromates contained in the final washings, which are too weak to boil down, is recovered, by the addition of sodium sulphide, as chromic oxide, which is press-filtered, mixed to a paste with lime, dried, and heated to low redness, until it becomes of a uniform yellow colour, and gives about 36 per cent. of chromic acid when dissolved in weak acid. In this state it is boiled with alkaline liquor in the same manner as in treating ore. This method of treatment is also adopted in working up the hydrated chromic oxide obtained by precipitation with magnesia in the manufacture of anthraquinone in coal-tar colour work.

When the quantity of chrome and lime mixture to be handled is large, the ordinary black-ash revolving-furnace of alkali works may be used for the roasting operation.

Electrolytic Oxidation of Chromic Oxide.—By electrolysis sodium chromate $Na_2Cr_2O_4$ is divided into sodium and hydrogen at the kathode and sodium chromate at the anode. In order to determine the amount of electrical energy necessary for this reaction, hydrated chromic oxide, dissolved in excess of caustic soda, was placed in a porous cell containing a platinum anode, and immersed in a glass vessel containing the kathode, an iron plate, both elec-

trodes being of the same size, namely, 7 by 12 centimetres. On closing the circuit, the initial tension of the bath rapidly fell from 12 to 5 volts, the quantity of the current, namely, 2 amperes, remaining steady during the two hours the experiment was continued. A constant and rapid evolution of hydrogen was observed at the negative electrode, which, however, as shown by the presence of caustic soda in the solution, was in part due to a secondary action (decomposition of water by the reduced sodium). At the positive pole, on the other hand, but little gas appeared, and the liquid in contact with it gradually assumed a yellowish tint. The chemical changes produced by electrolysis are expressed as follows—

$$Na_2Cr_2O_4 + 5NaHO.$$
$$3Na + 3H \text{ Kathode} \qquad \text{Anode } 2Na_2Cr_2O_4 + H_2O.$$

The experiment was stopped before the chromate was completely decomposed, when the liquor was found to contain 3,404 grams of sodium chromate, corresponding to an oxidation of 2·251 grams of chromate, or 0·563 gram per ampere-hour. Theoretically, 1 ampere-hour corresponds to a production of 0·298 gram of oxygen, or sufficient to oxidize 1·336 gram of sodium chromite.

$$Na2Cr_2O_4 + 2NaHO + 3O = 2Na_2CrO_4 + H_2O.$$

The useful effect realized was therefore $\frac{100 \cdot 0{\cdot}563}{1{\cdot}336} = 42$ per cent.

Electrolytic Production of Bichromate.—The apparatus used in the preceding experiment was charged in the anode cell with a solution of 58 grams of sodium chromate in half a litre of water, the kathode being immersed in pure water as before. As was expected, the bath showed a high resistance at first, which, however, soon diminished, the tension after half an hour being 8 and subsequently 6 volts for the remainder of the experiment, which lasted eight and a half hours, the volume of current fluctuating between 2 and 3½ amperes.

Oxygen and ozone were evolved at the anode, and hydrogen at the kathode, but there was no perceptible development of heat. The fluid in the anode cell was gradually reddened, while that in the kathode took a yellow tint from the diffusion of part of the neutral chromate, which was found to be present in the proportion of 0·4 gram to 14 grams of caustic soda. The anode solution when concentrated yielded more than 40 grams of sodium bichromate crystals, a very high proportion of the theoretical quantity, which in the 58 grams treated should be 50 grams, together with 14·8 grams of sodium hydrate.

Such portion of the neutral chromate as escapes decomposition by diffusion into the caustic soda liquor is not lost if the latter is returned to the process, and employed in washing out the roasted ore, in which case it returns to the concentration, and is electrolysed in a subsequent operation, so that there is practically no loss

of chromic acid; and as the half of the sodium in the neutral chromate is recovered in an active form, it seems likely that this method of forming the acid salt will supersede the souring process with sulphuric acid which necessitates the production of a large quantity of sodium sulphate, a salt that is comparatively worthless.

In conclusion, the Author points out that the electrolytic decomposition of alkaline chromates is not new, Buff having, as far back as 1856, subjected potassium chromate to the current of a galvanic battery, with the production of potassium and caustic potash at one pole, and chromic acid and oxygen at the other. The production of bichromate during the operation does not appear to have been observed on that occasion, and it is the formation of this intermediate product which constitutes the novelty of the present application.

H. B.

On the Condensation of Gases and the Direct Production of Zinc in the Blast-Furnace. By W. Hempel.

(Berg- und Hüttenmännische Zeitung, 1893, p. 355.)

The Author, after reviewing the methods proposed at different times for the reduction of zinc ores without the use of retorts, describes a number of experiments made in this direction by himself at the Technical High School in Dresden. It was supposed that metallic zinc might be separated from furnace gases in a melted form by subjecting the current to the action of a centrifugal machine at a temperature below the boiling-point of the metal; but a trial made at 470° Centigrade proved a complete failure, only zinc dust and oxide being produced, and other experiments showed that the nature of the product of ordinary distillation depended not merely on the nature of the gaseous atmosphere in which the zinc vapour forms, but that the degree of dilution is also of importance. Thus, in the reduction of zinc oxide in the Silesian or Belgian furnace, large quantities of zinc-dust are formed in the earlier stages of the distillation, liquid zinc being only obtained in a subsequent stage of the operation. This is due to the large quantity of indifferent gases given off by the charge at first, which carry away a considerable amount of zinc as vapour before it can coalesce into the liquid form. The received opinion that zinc oxide is only reduced at temperatures above the boiling-point of the metal is considered by the Author to be erroneous, and that it actually takes place at a strong red heat. The presence of uncondensable gases in large quantity being a necessary condition of blast-furnace working, the non-production of liquid zinc by the method of direct reduction is sufficiently explained. If, however, the reduction is effected in an atmosphere containing a minimum of carbonic acid, and the gases when cooled to 60° Centigrade are passed through a centrifugal machine, the zinc contained may be collected as dust

containing from 72 to 90 per cent. of metallic zinc, silver, lead, and copper in the ore being also volatilized as sulphides, but iron remains in the furnace. The Author considers, therefore, that the ordinary process of zinc-smelting might be modified as follows:—

1. The ore, whether calamine or blende, must be carefully roasted, to convert it as nearly as possible into oxide.

2. The zinc oxide mixed with three times its weight of bituminous coal and 5 per cent. of lime is coked, giving a material with 22·7 per cent. zinc, partly in metallic form.

3. The zinc coke is burned by hot blast in a closed top furnace, with a flue at the top connected with a centrifugal machine. The zinc volatilized and carried off by the gases is collected in the flues and the drum of the centrifugal machine. The gases when cleaned are utilized as fuel.

4. The zinc dust is subjected to a pressure of about 1,500 lbs. per square inch, which reduces it to about 10 per cent. of its original bulk, making it perfectly compact.

5. The compressed zinc dust is distilled in retorts without the addition of carbon, when about two-thirds of its weight of metallic zinc of great purity is obtained. Lead and silver if present remain in the fixed residue. This operation requires much less fuel for heating than the ordinary method of reducing, as the material in the retort being practically metallic zinc is a good conductor of heat when compared with the mixture of zinc oxide and carbon, which is a very bad one.

Probably a more complete reduction might be obtained electrolytically, as a lower tension current would be required than for the reduction of salts, the material being already in the metallic state to a great extent.

H. B.

Determination of the Velocity of Propagation of an Electric Perturbation along a Copper Wire by aid of a Method independent of all Theory. By R. Blondlot.

(Comptes Rendus de l'Académie des Sciences, Paris, vol. cxvii., 1893, p. 543.)

Two equal condensers of small capacity have their inner coatings connected to the exciting source, while their outside coatings are each divided into two independent portions, one of each being connected directly to two closely approximated discharging points, while the others are connected to the same points but through a considerable length of suspended copper wire. On the passage of a spark between the terminals of the exciting source, a spark will simultaneously pass between the discharging points, followed, after a short interval of time, by another spark which is caused by the disturbance traversing the wires. The difference in times between the two sparks is measured by photographic record on the

surface of a revolving cylinder in the usual way. Experiments over wires of different lengths gave a mean velocity of about 298,000 kilometres per second (given in the original as 298).

The results of Messrs. Fizeau and Gounelle in the year 1850, gave the figure of 177,000 kilometres per second, but this very low value the Author ascribes to the prolonged duration of the exciting spark, which was about three thousandths of a second, whereas in these experiments the duration of the spark was of the order of one-hundred-millionth of a second.

F. J.

Absolute Measurement of the Flow of Electricity from Points.

By JULIUS PRECHT.

(Annalen der Physik und Chemie, vol. xlix., 1893, p. 150.)

The general phenomena of the dissipation of electrical charges by points have been long known; but in spite of the importance of the subject in relation to lightning conductors, few observations have been made from the quantitative standpoint. The Author has attempted to determine at what difference of potential the discharge commences, and having begun, at what minimum voltage it ceases. He also gives some details as to the quantity of the flow. The apparatus was arranged as follows:—A Leyden jar, which could be charged by an influence machine, was furnished with a hollow brass ball in place of the usual knob. A stout brass wire, insulated by shellac, supported a small aluminium-leaf electroscope in the centre of this sphere, and carried at its upper extremity a socket for the insertion of the point to be experimented with. This wire was connected, through a very high resistance, with the sphere, and another smaller resistance was inserted between the Leyden jar and a conducting wire, 3 metres long, leading to the influence machine. By this arrangement the jar was slowly charged, and the leaves of the electroscope remained stationary until the discharge from the point commenced, when the sudden change caused a difference of potential between the brass wire and the hollow sphere, sufficiently marked to make the aluminium leaves diverge before it could be equalized by the flow of electricity through the high resistance. The movement was observed through apertures left for the purpose. The resistances employed consisted of glass tubes filled with cotton thread and furnished at each end with wires. The cotton was moistened by breathing on it until the conductivity was found to be sufficient, and the tubes were then sealed.

The actual difference of potential at the moment of discharge was measured with a quadrant electrometer, carefully protected from the direct influence of the rest of the apparatus.

The following are some of the results. When the same point

was used many times in succession, the potential difference of discharge was found to increase, reaching a maximum at about the eighteenth experiment of nearly 25 per cent. above its original value. This effect was ascribed partly to alteration of the apex and partly to the charging of the surrounding bodies.

When the air was full of dust an increase of potential difference —sometimes 10 per cent. above the normal—was necessary for the flow to begin. Allowing gas to burn in the neighbourhood of the point produced a similar and even greater effect, which persisted after the flame was put out, but was got rid of by thoroughly ventilating the room.

The effect of ultra-violet rays was studied. With positive charges in sunlight from an open window a potential difference of 5,380 volts was required, and in the dark 5,330 volts; but with negative charges the figures were, in sunlight 3,680 volts, and in the dark 3,860 volts. An induction spark between zinc poles gave similar results, and the effect was at once lessened by the interposition of a piece of glass between the quartz lenses used to concentrate the light. Long-continued positive discharges from a point sometimes resulted in the formation of a crater-like cavity, but no such effect was observed with negative electrification. With very sharp points it was often noticed that the electroscope gave several slight movements before the definite discharge began. For example, a point made of sheet aluminium gave indications at + 1,871 volts and – 1,065 volts, but the true flow of electricity only began at + 4,173 volts and – 2,971 volts. The potential difference of discharge appears to be conditioned by the entire configuration of the pointed conductor, and not merely by the shape of its apex, but the Author was unable to find any simple law expressing this relation. Although in the majority of instances the positive values were greater than the negative, this was by no means invariably the case. Electricity escapes less easily from a bundle of points than from a single one. Thus with twenty needles the electromotive force of discharge was + 7,600 volts or – 5,700 volts, while with half that number the figures were + 5,600 volts and – 4,200 volts. With five needles together they were + 4,900 volts and – 3,200 volts, the average for a single needle being + 4,300 volts and – 3,000 volts. Sharp edges appear to act in a similar way, *i.e.*, the shorter they are the lower is the potential difference of discharge.

The minimum potential, viz., that at which the outflow ceases, is always less than that at which it began, the difference being sometimes as great as 1,000 volts. It depends, however, largely on the extent to which the surrounding air has been electrified in the course of the experiment.

The relation between the electromotive force and current of the discharge from a point was studied by plotting the results in graphic form. The curves were parabolic in character, the ordinates representing the current increasing at first slowly and afterwards nearly in a constant proportion to the voltage. The

quantity of the discharge from a negative point is as a rule greater with the same potential difference than from one positively electrified. Sharp points give also larger currents than blunt ones, but with a very high electromotive force the difference is not so marked. Experiments made on six different occasions show that to produce a discharge of 100×10^{-9} amperes from one particular needle an average potential difference of 8,188 volts was required.

The Paper concludes with some calculations as to the quantity of electricity which may be dispersed silently by an ordinary lightning conductor during a storm.

G. J. B.

Constant-speed Series-wound Motor. By G. Mariotti.

(Elektrotechnische Zeitschrift, 1893, p. 557.)

Series-wound motors are preferable to shunt or compound-wound motors wherever the load is not subject to wide variations. If, however, the load on a series-wound motor be liable to considerable reduction, there is danger of the motor racing, and then the shunt motor, or one with compound winding on the field magnets, is preferable. The Author describes a method of exciting a series-wound motor, under which the speed for all loads remains approximately constant. The method consists in supplementing the exciting current by means of a small overcompounded dynamo driven by the motor. The terminals of the auxiliary dynamo are coupled to the terminals of the field coil of the motor, and the characteristic of the dynamo is so chosen as to increase the auxiliary current when the load on the motor decreases, thereby strengthening the motor-field and keeping the speed of the motor within the desired limit. To prevent reversal of polarity, due to overloading, an automatic magnetic cut-out is employed. The Author gives the theory of this apparatus by means of various diagrams, and refers to a practical application of his method on a 14-HP. motor, working on a 500-volt circuit. The output of the auxiliary dynamo in this case is 400 watts.

G. K.

The Design of Alternating-Current Motors. By E. Kolben.

(The Electrical World, New York, vol. xxii., 1893, p. 284.)

The Author gives rules for the designs of single phase motors, but states that they are also applicable to motors of the multiphase type. The stationary or inducing part consists of a cylinder of iron built up of disks, and the revolving or induced part, called the armature, is a cylinder revolving within the fixed part, the

clearance being as small as possible. The field-winding is placed in holes very near the inner circumference of the stationary part, and the armature winding is similarly arranged on the rotating part. The Author points out that a high frequency requires the field-winding to be arranged so as to form a large number of poles around the inner circumference of the field, or otherwise the speed would become inconveniently large. The result of having many poles is that the magnetic resistance in the air-gap between the fixed and rotating part becomes large, and this increases the magnetizing current. For those reasons the Author prefers a low frequency. The armature may be wound in sections, so as to produce alternate poles, or the winding may consist of bars short-circuited by stout copper rings at both ends. The latter is treated as a special case of the former system of winding. As regards induction in the iron parts of the machine, the Author recommends the following values according to the frequency ~

~	B
40	6,500 to 5,500
50	6,000 „ 5,000
60	5,000 „ 4,500
80	4,500 „ 4,000
100	4,000 „ 3,500

The induction in the air-gap should be between 2,000 and 3,000, when the exciting power required for the field will vary from 250 to 380 ampere turns per inch of circumference. The air-gap varies from 0·04 inch to 0·08 inch according to the size of the machine. In determining the power of the motor the Author starts with calculating the power wasted in the armature winding at a given magnetic slip $\frac{1}{s}$. The speed at which field is cut by the winding is then proportional to $\frac{s-1}{s}$, and the total power delivered is s times as great as that wasted in heating the armature winding. Formulas are given for the total power and the various losses whilst the determination of the currents for the motor running idle and loaded is made graphically. An example is also given showing the efficiency of a 2-HP. motor running at 42 and 50 cycles under varying load.

G. K.

The Measurement of Power in Three-Phase Circuits.

By Dr. O. Frölich.

(Elektrotechnische Zeitschrift, vol. xiv., 1893, p. 574.)

The Author deals with circuits grouped in the manner known as the link connection. If A B C are the currents in the three circuits and P_1 P_2 P_3 the values of the absolute potential at the

respective points where the power is delivered, he finds that the total power is given by the expression

$$W = A(P_1 - P_3) + B(P_2 - P_3),$$

or

$$W = B(P_2 - P_1) + C(P_3 - P_1),$$

or

$$W = C(P_3 - P_2) + A(P_1 - P_2).$$

If each of these three points be joined by a wire of resistance with a junction common to all, we obtain a star connection and the sum of the currents in the three wires is zero.

$$i_\alpha + i_\beta + i_\gamma = 0.$$

Since $(i_\alpha - i_\beta)\, r = P_1 - P_2$, and similarly for the other circuits,

$$\frac{W}{r} = A\, i_\alpha + B\, i_\beta + C\, i_\gamma$$

This formula shows how the total power is divided between the three circuits and might be used to measure the power separately for each. For this purpose a watt-meter must be inserted in each circuit, the movable coil being traversed by the main current and the fixed coil by the current i. If the total power only has to be measured, a single instrument may be used containing three fixed and three movable coils, the latter being of course fixed relatively to each other.

G. K.

Notes on Multiphase Alternate Currents.

By G. de Chasseloup-Laubat.

(Mémoires et Compte rendu de la Société des Ingénieurs-civils, August, 1893, p. 168.)

This Paper is a critical review of the various possible methods of transmission of electrical energy with a more detailed examination of the method indicated in the title.

A comparison of the advantages and disadvantages of continuous and alternate currents leads to the conclusions:

(1) That alternate currents are preferable for long distance transmission both by reason of the higher tensions available with them and by reason of the facility of transforming them to lower tension.

(2) That continuous currents are preferable for the service distribution, because accumulators can be used with them, from which results a better utilization of the machinery and greater safety.

To satisfy these conclusions it became necessary to transform the energy of an alternate current into that of a continuous current, a problem which is solved by any satisfactory alternate

current motor, since such a motor may be used to produce a continuous current.

Alternate-current motors may be constructed with a magnetic field, either constant, alternating, or revolving. The first of these three classes of motors has been much treated of and is not here further discussed. The second has a very low efficiency which shuts it out from practical use. The third class is discussed at considerable length by the Author. The intensity of the revolving field is calculated for two-phase, three-phase, and multiphase currents, and the mechanical work done when an armature revolves in that field by reason of the currents induced in it. The losses are investigated due to the resistance of the inducing system, to hysteresis and Foucault's currents, and to the resistance of the armature coils. The coefficient of self-induction of a turn of the armature coils is calculated, and also of the existing circuit, and the method of calculating the elements of a motor is discussed.

A further chapter is devoted to machines for directly transforming alternate currents into continuous currents, as the three-phase machine of Dobrowolski and the machine of Messrs. Hutin and Leblanc.

The Paper concludes with a bibliography of the subject.

C. F. F.

The New Cail-Helmer Alternator.

(L'Industrie Électrique, 1893, p. 470. 8 Figs.)

The Author states that the electrical branch of the Société des Anciens Établissements Cail was only started about three years ago by Mr. Bourdet, and the management is now in the hands of Mr. Helmer. After completing their designs for direct-current plant, the company has now produced new types of alternators, transformers and other accessories. The Author considers that the special features of the new machines are, simplicity of design, special form of the magnetic circuit, and subdivision of the parts subjected to variations of induction, so as to almost wholly suppress the Foucault currents. The armature is stationary and the field-magnets revolve. The special description given in the Paper refers to the 25-kilowatt type, which runs at 600 revolutions per minute and has a frequency of sixty alternations per second. The fixed armature consists of twelve coils placed in the interior of a framework consisting of two rings held apart by stays and mounted upon the bed of the machine.

The field-magnets are twelve in number also and attached to two star-shaped carriers, which can be adjusted for position on the shaft by means of nuts. The cores of the field-magnets and armature-coils are entirely composed of rectangular plates of very

soft wrought-iron; these plates are bent into the form of the letter U. Each core is formed of two sets of these bent plates placed side by side in the same plane. The use of rectangular plates enables the cores to be produced much more cheaply than in most of the other types of alternators. Special pillar-studs, screwed at both ends, are used for holding the cores of the armature-coils in position. One end of the stud holds the core against a non-magnetic base, while the other is attached to the cross-bar, which is joined to the two solid ring-frames. The nuts which hold the armature cores act upon stirrup-pieces, which keep the plates apart inside. The armature-coils surround the inner pole pieces and rest in these stirrups; they are held by bolts and are well insulated by plates of non-magnetic material. In a similar way, the field magnet-cores are held in position by bolts, which retain the stirrups in which the coils rest, and the coils are held by other stirrups, which are fixed on the first stirrups by means of studs.

This arrangement has been effected with a view of keeping the active parts of the machine as far as possible from those which are inactive. The magnetic circuit is made as short as possible, and a very small air-space is obtained, so that the loss for exciting the machine is only 2 per cent. The cores of the field-magnets and of the armature-coils are of equal section. In order to diminish the fluctuations, the air-gap becomes gradually wider from the centre of the pole-piece to the edges in the direction of rotation. By reason of the excess of the magnetization of the field-magnets over that of the armature, the fall of potential is very small when the load is increased. Lubrication is effected by means of loose rings, which dip into oil-vessels and are carried round as the shaft revolves. The height of oil in the vessels is visible by means of a glass gauge. It is essential that the machine can be easily examined for repairs, and in order to facilitate this, the bed of the machine is provided with a dovetail groove into which a plate can be slipped, and then one of the bearings can be moved outwards sufficiently far to allow the armature to be entirely clear of the field-magnets. The following Table of details is given:—

Total output	25 kilowatts.
Useful difference of potential between terminals	2,400 volts.
Revolutions per minute	600
Number of armature coils	12
Frequency in alternations per second . . .	60
Maximum induction in the iron of the armature in C.G.S. units	5,070
Weight of the armature-windings	147·4 lbs.
„ „ field-magnet windings . . .	198·0 „
„ „ iron core in armature	389·4 „
„ „ „ „ field-magnets . .	620·4 „

The Author then remarks that, if dynamos are driven by belting, attention is necessary to maintain a proper degree of

tension of the belts, otherwise either there will be slipping or the bearings will be heated, and, of course, belting occupies a great deal of space. In order to obviate these disadvantages, the Société Cail construct a machine which they term the fly-wheel alternator. In this case the fly-wheel is itself the framework upon which the field-magnets are fixed, and it is driven by either a Corliss or Sulzer type of steam-engine. The principle of construction is similar to that of the alternator already described. The field-magnet coils are formed of thin bent plates, and these are forced side by side into the grooved rim of the fly-wheel and held in position by the stirrups already referred to. The coils are placed inside these stirrups. Similarly, the cores of the armature-coils are forced into a grooved frame, which is concentric with the fly-wheel. The armature-frame rests upon shoes which fit into slides upon the bed-plate, and thus it is very easy to move the armature aside in order to inspect the windings.

Transformers of very simple type are employed with these machines. The outer part of the magnetic circuit consists of a number of rings of thin plates held together by bolts between two rings of cast-iron. The internal part of the hollow tube thus formed has recesses slotted in it at equal distances from each other. These recesses are for the purpose of receiving the bobbins. The end plates of cast-iron are accurately turned to a standard size to receive the internal part of the transformer. The central part consists of a cored centre, upon which are placed a number of star-shaped plates; these plates are held tightly together by means of bolts and circular end-plates, which are turned to fit accurately into the outer rings. The star-shaped plates fit easily into the outer ring-plates, so that there is a slight air-gap, thus forming a break in the magnetic circuit.

The central cylinder can be rotated upon a pivot fitting into a recess in the bottom plate, so that the magnetic resistance varies from a minimum to a maximum corresponding to a position distant from the minimum of a number of degrees, half of the angle between two adjacent prominences on the central part; thus, for a transformer with a six-pointed centre, the distance between the points of maximum and minimum resistance would be 30°. This type of transformer can be taken to pieces very easily, as the internal part can be lifted out bodily with all the windings. The primary and secondary windings are wound in pairs close together upon each arm of the central star. A bi-polar type is only used for low powers; but for larger powers multi-polar types are always employed.

The following Table refers to a transformer for 24 kilowatts:—

Output in kilowatts	24
Frequency in alternations per second . . .	60
Maximum induction in C.G.S. units . . .	5,000
Sectional area of the magnetic circuit outside the centre cylinder	54·25 sq. ins.
Weight of wrought-iron	583 lbs.

PRIMARY WINDING.

Useful difference of potential between the terminals	2,400 volts.
Number of coils	512
Weight of wire	98 lbs.

SECONDARY COIL.

Useful difference of potential between terminals	120 volts.
Number of coils	26
Weight of wire	100·75 lbs.

E. R. D.

Voltmeters for Alternating-Current Stations. By B. SZAPIRO.

(Elektrotechnische Zeitschrift, 1893, p. 466.)

It is well known that a voltmeter of the electro-magnetic type alters its calibration with an alteration of frequency, and for this reason it is customary to state the frequency for which the instrument has been calibrated. There is, however, also another cause affecting the calibration, namely, the form of the curve representing the variation of electromotive force. The Author shows that in a voltmeter of the hot-wire type, a departure from the true sine-curve does not affect the correct reading of the instrument, but, where the voltmeter is of the electro-magnetic type, the opposition it offers to the passage of the current depends on the shape of the electromotive-force curve, the effect being the more marked the smaller the ohmic resistance of the instrument is in comparison to its self-induction. The Author gives the example of a voltmeter of 3,000 ohms resistance and 1 henry inductance, which had been calibrated on a sinusoidal electromotive force, and was then used on a Ganz alternator, the curve of which departs considerably from the true sine-curve. The calibration in this case was wrong by 8 per cent., that is to say, the same effective electromotive force when produced by a machine having a true sine-curve will give a reading higher by 8 per cent. than when produced by a Ganz machine. If this voltmeter were used in a central station, equipped with the latter type of machines, there would be danger of overstraining the lamps, and the Author considers it therefore necessary that voltmeters should be calibrated not only at the frequency at which they will be used, but also on a circuit supplied by the same type of machine as those in the central station for which they are intended.

G. K.

Arc- and Glow-Lamps. By A. Bainville.

(L'Electricien, 1893, p. 97.)

In this article the Author compares the cost of lighting by means of glow-lamps and arc-lamps under different conditions of supply. He distinguishes between private generating stations where the cost of power is low, and central stations where it is generally much higher, taking the unit at 0·5*d.* and 3*d.* in the first case, and 6*d.* and 1*s.* in the second case. In making an estimate of the cost of arc-lighting he assumes the expense for carbon to vary from 0·11*d.* to 0·45*d.* per hour, according to the current employed, the extreme limits being 3 amperes and 20 amperes. The annual cost for renewals and repairs to arc-lamps he takes at 30 per cent. on the cost of the lamp, this being estimated at £6.

The labour required for cleaning and trimming the lamps he assumes at one hour for each lamp per day.

A Table is given showing the cost per spherical candle-hour calculated on the above basis. As regards lighting by glow-lamps the Author distinguishes between four different cases when lamps are employed requiring 2, 2½, 3 and 4 watts per candle, having a life of 150, 300, 600 and 1000 hours respectively. The resultant costs per spherical candle are given on the supposition that the light is required on an average for six hours per day. In comparing the results he finds that where the power can be obtained at the low rate of ½*d.* per unit the 3-watt glow-lamp is under all circumstances more economical than the more efficient glow-lamp or arc-lamp. Where the power can be obtained at 3*d.* per unit glow-lamps consuming from 2 to 2½ watts per candle are the most economical. With power at 6*d.* per unit the arc becomes more economical than the glow-lamp when the light required from each exceeds 300 candles, and at 1*s.* per unit the limit is 150 candles, the most economical type of glow-lamp to be used being that which requires 2 watts per candle. The general result of the Author's investigation is that under nearly all circumstances the glow-lamp is preferable to the arc-lamp.

G. K.

New Investigations of the Alternate-Current Arc.

By A. Blondel.

(La Lumière Electrique, vol. xlix., 1893, pp. 501 *et seq.*)

These investigations have been conducted by stroboscopic methods, so that the behaviour of the electromotive force, current, and arc are exhibited for each integral part of one alternation. The usual arrangement of a movable contact was adopted, with

photographic registration of the indications of the different instruments, which measured the electrical quantities involved; and, as the sensitized paper covered the periphery of a drum mounted on the same axis as the movable contact, the deflection of the instruments parallel to the axis of the drum, and the angular position of the contact or drum, determine the ordinates and abscissas of the curve, which can be traced for a complete period by a slow movement of the contact point through the requisite angular distance. The actual arrangement of the apparatus is fully described, as well as the preliminary determination of the constants; while the damping of the instruments was such that a complete curve corresponding to a single period could be recorded in two minutes with a limit of error certainly under 1 per cent.

Five different forms of alternators were used in the course of the experiments, but the majority of the results were obtained from a Siemens machine of very low resistance and inductance, because preliminary determinations showed its curve of electromotive force to be almost perfectly sinusoidal; though as regards its effect on the tension required for maintaining the arc the form of the curve of variation of electromotive force was found to be practically negligible.

The Paper is copiously illustrated with reproductions of the photographic records, which furnish for each observation the instantaneous values, for one complete period of an alternation, of the following three quantities: first, the electromotive force at the terminals of the alternator, or what may be termed the available electromotive force; second, the difference of potential between the carbons or the tension of the arc; and, third, the current traversing the arc.

The first of the above quantities will be naturally a function of the conditions of the whole circuit, and the effect on the arc itself of the rest of the circuit is thus clearly exhibited. For instance, with a non-inductive circuit, the tension between the carbons during the period of extinction of the arc coincides with that between the terminals of the machine until the arc is re-established by a species of disruptive discharge, the tension then being somewhat higher than that required for maintaining the arc, this latter tension varying with the length of arc and the quality of the carbons; the current also remains nil for a considerable time, especially with arcs of low intensity and in the hissing state.

This fact reveals the necessity of having the available electromotive force relatively high, and thus the requirement for arcs of mean intensity that resistance or inductance should be included in the circuit. With the hissing arc, however, the tension between the carbons is between 25 and 30 volts, and almost constant for the whole time between the extinctions.

With an inductive circuit, the current and electromotive force at the arc lag behind that at the machine, this lag being considerably reduced during the periodical extinction of the arc, so that the reversal of the electromotive force at the arc is the more

abrupt the higher the inductance of the circuit, while, on the contrary, the duration of zero current almost or completely vanishes; the extinction of the arc, however, still maintains, as proved by subsequent experiment, so that the current must be conducted by heated gases, which are very weak in luminous rays, and to the fact of increasing the conductivity of the heated gases must be ascribed the advantage resulting from the use of cored carbons.

To determine the points of extinction and re-establishment of the arc, similar curves were obtained by projecting the image of the arc from a mirror, fixed to a pendulum controlled by the current, on to a revolving drum; and these show the luminosity of the arc varying continuously until it is extinguished just before zero current is reached, and not starting again until the current has attained a considerable value. This seems to prove that the principal luminosity of the arc is the result of the molecular dissociation of the carbon, which does not occur until a definite density of electric current is reached.

The power-factor, or ratio of real to apparent watts, approaches unity according as the arc is quieter, and the core of the carbon softer. With cored carbons and low voltage, it reaches, and even exceeds, 0·95; and the arc is then similar to an inductionless resistance, while with solid carbons and a roaring arc the factor may be as low as 0·70 at frequencies of twenty-six and fifty-two per second; the forms of the curves with the longer or shorter extinction of the arc are quite sufficient to account for this variation without any necessity for ascribing counter-electromotive force or inductance to the arc, the existence of either of which must be considered as absolutely negatived, as also the idea of preventing hissing, by adding electrostatic capacity to the arc. The variation of current with length of arc is not a constant, but depends to a slight degree on the form of the periodic curves, the principal drop of potential being between the positive carbon and the arc. Alteration of frequency has no effect in changing the character of the curves, but may have some slight influence on the power-factor and stability of the arc.

The stability, as may be expected, depends principally on the nature of the carbons, and the available electromotive force, though it is somewhat favoured by increased frequency of the alternations and inductance of the circuit. The employment of arcs in series is advisable from every point of view, but does not necessarily entail the use of low voltages, for the adoption of carbons with very soft cores transforms the alternating arc into a sort of mixed incandescent lamp, the stability and power-factor of which are known to be excellent, though the luminous efficiency is only moderate.

F. J.

The Erding Central Station.

(Elektrotechnische Zeitschrift, 1893, p. 558.)

This is one of the first installations in Germany in which three-phase alternating-currents are used for lighting and power purposes. The generating-station is about 2 miles from the town of Erding (Bavaria), where the water-power of the river Sempt has been utilized for driving, by means of a turbine, two three-phase generators each of 60 HP. Each machine with its exciter is mounted on one bed-plate, and the necessary synchronizing devices are provided for coupling the machines in parallel, in case it should be necessary to change over from one machine to the other. The station voltage is regulated by a rheostat in the shunt of each exciter, and varies from 1,500 to 1,575 volts, according to the output. The line consists of three bare copper-wires, carried on oil-insulators on wooden posts, the distance between any two wires being 28 inches. To protect the line against lightning a barbed wire is carried over the top of the poles and connected to earth at every fifth pole. There is also a lightning-protector, with an automatic interrupter midway between the generating-station and the town. The distributing-plant in the town consists of five transformers, which are placed on high masonry pillars. All five transformers are coupled in parallel, both on the primary and secondary side. The distributing-mains, with 115 volts pressure between any two circuits, are also carried overhead. The consumers pay for each lamp installed at the rate of about 1*s*. per candle-power per annum, and the current is supplied for power purposes at the following annual rates: £3 10*s*. for a motor of ½ HP.; £6 10*s*. for a motor of 1 HP.; £12 for a motor of 2 HP.; £24 for a motor of 5 HP.

G. K.

Recent Practice in Telegraph- and Telephone-Line Construction of the German Administration. By — Kohlmann.

(Archiv für Post und Telegraphie, 1893, p. 687.)

This Paper contains fully detailed particulars of the construction of the posts, insulators and wires for the overhead wires, and cables for the underground wires, for both Telegraphic and Telephonic purposes, as well as specification of the materials employed. Wooden poles are generally adopted, iron ones being used only where necessary for telephonic conductors; the cross arms of a length to carry the necessary number of insulators are, however, of iron, being constructed each of two bars about 2-inch by ½-inch section, spaced 2 inches apart by the necessary distance pieces and rivets, to which the insulator stalks are bolted in the

ordinary way. The cross arms for the telephonic conductors are similar but of lighter section. As regards the iron wires for telegraph purposes the tensile strength (40 kilograms per square millimetre, twenty-five tons per square inch), ductility by torsional and bending tests, and conductivity (13 per cent. of pure copper), as well as the test for quality of galvanising, are worked out and specified for each size of wire, which varies from 6 to 1·7 millimetre (0·240 to 0·068 inch); for telephone wires bronze is adopted and the particulars also recorded in detail.

In the underground cables fibrous insulation protected by lead is gradually replacing gutta-percha, and hoop iron armouring the previous wire sheathing; while for telephonic cables paper-insulation with air space is being adopted, on account of the low electrostatic capacity resulting therefrom.

F. J.

Long-Distance Transmission of Power. By E. Arnold.

(The Electrical World, New York, vol. xxii. 1893, p. 58.)

In this article the Author discusses the influence of the static charge in long-distance power lines, and gives the results of a series of experiments carried out with the three-phase transmission plant between Hochfelden and Oerlikon, in Switzerland. The distance of transmission is 14½ miles, and the plant is of similar construction to that used for the Lauffen-Frankfurt transmission, where the static induction has never given rise to any difficulties even with the most varying atmospheric conditions. The generators at Hochfelden deliver 300 HP. at 50 volts and 3 × 1,400 amperes. The current is transformed up by three-phase transformers, having a ratio of 1 : 154·6. The line consists of four wires of 16·0 millimetres diameter, run on poles 25 feet high, and at an average distance of 17·7 inches from each other. In order to determine the influence of the static charge the delivery ends of the line were disconnected, and the generators were run both with variable speed and constant pressure, and with constant speed and variable pressure. The frequency was varied from 49·3 to 21·6, and Tables are given showing the low-pressure and high-pressure currents under these varying conditions. Generally speaking, when the frequency is constant the secondary or high-pressure current, which in this case is entirely due to the electrostatic capacity of the line, rises with the pressure, whilst at constant-pressure it rises as the frequency is increased. The same tests were repeated when the line was entirely disconnected, and the results are shown plotted in curves, from which it is seen that the primary current, both for the line connected and disconnected, is directly proportional to the pressure. One diagram is given which shows that the curve of primary current when the line is disconnected intersects the corresponding curve when the line

is connected, from which the Author concludes that the capacity of the line tends to diminish the primary current at the generating station, and in certain cases actually does so. This point he then investigates theoretically, and he finds that by the addition of self-induction to the line it is always possible to so balance the effect of capacity as to work without any phase-difference between current and electromotive force.

The practical conclusion of this investigation is that the capacity of the line need never interfere with the proper working of a long-distance transmission plant.

G. K.

The Electrical Portage between Lakes Ontario and Erie.

By T. C. Martin.

(The Electrical Engineer, New York, vol. xvi., 1893, p. 121.)

By concession of the Canadian Government, and an annual royalty of £2,000, the route followed by this railway closely skirts the brink of the Niagara river for the whole of the track, and thus affords an admirable means of seeing all its beauties. The line, 13 miles in length, commences at Queenston on Lake Ontario, where connection is made with the Niagara Navigation Company's steamers, and terminates at Chippewa on Lake Erie, where a connection is made with the Michigan Central Railroad, and an intermediate connection with the Grand Trunk, and as the line has been constructed on the standard gauge of 4 feet 8½ inches, facilities are thus furnished for through traffic. The whole line, 37 per cent. of which is on curve, is solidly built and well ballasted, and the poles carrying the trolley wire are either of steel or wood according to æsthetic requirements. The maximum gradient is 5 per cent., which prevails for about 1½ miles from the Queenston end, the rest of the track being nearly level. Three regular bridges are crossed, and a trestle over a ravine 500 feet long and 135 feet high. The power house, a substantial stone building, 100 feet by 62 feet, is situated at a distance of about 2½ miles from the Chippewa end, *i.e.*, close to the Falls, and contains two turbines, each of 1,000 HP. with space for a third, which drive at present two dynamos of about 800 HP. in total. To meet special emergencies of heavy traffic up the Queenston incline, it was found more economical to provide a supplementary steam-power house at that end, feeding that portion of the line separately, than to transmit the extra current from the Falls, but this steam-power is generally completely shut down, and the whole line is worked throughout by the hydraulic plant.

The total cost up to the present has been about £120,000, and the installation appears to be giving every satisfaction to the proprietors. The rolling stock consists of four closed and eighteen open motor cars, each fitted with two motors, and in addition

eighteen open cars as trailers without motors. The return fare over the whole distance is three shillings, and the traffic is very heavy. The Author states that he spent one complete day on the line, during the whole of which he never saw a lightly loaded car, and on the same day 17,126 passengers were carried during the fifteen hours the line was open.

The Paper is fully illustrated with plans and views of the line, power-house, &c., but only a few details of the equipment are given.

F. J.

Report of Committee on "The Return-Circuit of Electric Railways." By T. J. McTighe.

(The Electrical Engineer, New York, vol. xvi., 1893, p. 282.)

In a paper read before the New York State Street Railway Association, the Author formulates the following requirements for securing the best return, namely, that the intrinsic resistance should be found by the rails themselves, and be low enough to require no help from the earth. The rail-bonds should be as heavy and short as practicable, consistent with due allowance for expansion and vibration, formed of a single piece with integral rivets, tightly closed in holes freshly reamed at the moment, but placed so as to be convenient for inspection, and protected by some application of shellac or asphaltum against corrosion. A liberal addition of cross-bonds should also be introduced. If the line be not connected direct to the power-house, then a return-conductor, which may be more economically and efficiently formed from old rails bonded as above, should be laid underground, and be of a resistance low enough to carry the maximum current with only nominal drop of potential. Illustrations of how it is at present usually done are given, with remarks as to the want of economy and failure to secure the requisite low resistance; and the method adopted by the Author of securing efficient bonding is described with some detail.

F. J.

Street-Railway Rheostats as Car-Heaters.

By W. E. Harrington.

(The Electrical World, New York, vol. xxii, 1893, p. 208.)

The Author records the results of experiments which were undertaken with a view to determine the practical advantages of heating electric tramcars by means of the rheostats required in the working of the motors. In the first test the car was kept in a shed, and the switch was handled as nearly as possible the

same way as would obtain on the road. The experiment lasted over five hours, during which time the temperature inside the car rose 12° Fahrenheit over the external temperature. The same car was then run on the road in between the other cars in service, the doors and windows being kept closed. Under these conditions, with the speed regulation entirely rheostatic, a rise of temperature not exceeding 7° Fahrenheit was observed. The Author draws two conclusions from these experiments: firstly, that the use of rheostatic coils to serve also as car-heaters is not of sufficient practical advantage to warrant the adoption of the special arrangement which would be required for this purpose; secondly, that rheostatic regulation is not nearly so wasteful as commonly supposed, and is on practical grounds to be preferred to cumbersome and complicated commutation methods.

G. K.

Electro-Magnetic Properties of Parallel Conductors.

By C. P. Steinmetz.

(Elektrotechnische Zeitschrift, 1893, p. 476.)

In this article the Author refers to an investigation in hydraulics relating to the flow of an incompressible liquid between two surfaces, which is applicable to the flow of electric current and electro-magnetic induction. Starting from the well-known theorem that the streams-lines are everywhere intersected at right-angles by equipotential surfaces, he shows how one system of conductors can be replaced by another system, either of similar form and different dimensions or according to the theorem of reciprocal radii by systems of different form. By applying this theory to the case of electric conductors he reduces the problems to simple equivalent forms which can be easily treated.

Thus a system composed of a straight wire and parallel plane (overhead lines and surfaces of the earth) he transforms into a system of two concentric cylinders, and in the same way two parallel conductors can be transformed into a system of straight wire and parallel plane, or two concentric cylindrical conductors.

In the former case the surface of the earth, as far as its influence on the capacity and electro-magnetic induction is concerned, is equivalent to a concentric cylinder having a radius equal to twice the height of the wire over the earth. For two parallel wires having the radii r_1 and r_2 and distance d between their centres, a system of concentric cylinders of radii r and R may be substituted, provided it satisfies the following formulas:—

$$\frac{R}{r} = m + \sqrt{m^2 - 1}$$

$$m = \frac{d^2 - r_1{}^2 - r_2{}^2}{2\, r_1\, r_2}.$$

If both wires are of the same size and their radii small as compared to their distance the formula becomes :—

$$\frac{R}{r} = \left(\frac{d}{r}\right)^2.$$

The coefficient of self-induction for two concentric cylinders of radii R and r and length l is given as

$$L = 2\,\mu\,l\,10^{-9}\log\text{nat}\,\frac{R}{r}\ \text{Henry,}$$

where μ is the permeability of the medium, being unity in case of air.

The capacity of the same system of conductors in microfarads is given as—

$$K\,m = \frac{1{\cdot}11\cdot 10^{-6}\cdot\epsilon\cdot l}{2\log\text{nat}\,\frac{R}{r}}\ \text{microfarads,}$$

where ϵ is the specific inductive capacity of the medium.

In treating practical cases the system of conductors (either wire and earth or two wires) is resolved into the equivalent system of concentric cylinders and their capacity and self-induction determined by the above formulas.

For convenience of calculation, the Author gives the following Table not only for self-induction and capacity, but also for magnetic resistance and electric resistance where the wires are immersed in a badly conducting medium :—

—	Cylindrical Wire of Radius r and length l at Distance d over Surface of Earth.	Two Cylindrical Wires each of Radius r and length l at Distance d apart.	—
Electric resistance of intervening medium	$\frac{\rho}{2\pi l}\log\text{nat}\frac{2d}{r}$	$\frac{\rho}{\pi l}\log\text{nat}\frac{d}{r}$	Ohm.
Magnetic resistance of intervening medium	$\frac{2\pi}{\mu\log\text{nat}\frac{2d}{r}}$	$\frac{\pi}{\mu\log\text{nat}\frac{d}{r}}$	C. G. S. units.
Coefficient of self-induction . . .	$2\times10^{-9}\mu\,l\log\text{nat}\frac{2d}{r}$	$4\times10^{-9}\mu\,l\log\text{nat}\frac{d}{r}$	Henry.
Electrostatic capacity	$\frac{1{\cdot}11\times10^{-6}\epsilon\,l}{2\log\text{nat}\frac{2d}{r}}$	$\frac{1{\cdot}11\times10^{-6}\epsilon\,l}{4\log\text{nat}\frac{d}{r}}$	Microfarads.

G. K.

Experiments on the Resistance of Air and other Gases to the Movement of Bodies. By L. CAILLETET and E. COLARDEAU.

(Comptes Rendus de l'Académie des Sciences, Paris, vol. cxvii., 1893, p. 145.)

The Authors in a previous communication have described their attempts to study this question by direct rectilineal motion from the Eiffel Tower; but the variations in the atmospheric conditions having introduced disturbing causes, they here describe the results obtained from circular motion in a large closed chamber in which the pressure could be raised up to eight or ten atmospheres. The disposition of apparatus comprised a cord and weight which in falling revolved an arm to which was fixed a large pallet, the whole being controlled from the exterior without disturbing the condition of the pressure of the gas. The results of several series of experiments show that the resistance opposed by a compressed gas to the movement of a plane is proportional to the square of the velocity of the said plane, and directly proportional to the pressure and density of the gas, or

$$R = K\,S\,D\,P\,V^2,$$

where if S, the surface, be given in square metres, P the pressure in atmospheres, V the velocity in metres per second, and D the density referred to air as unity, K in kilograms is 0·07, and is constant for values of V between 2 and 25 metres per second in British units of lbs. and feet K = 0·00133 lbs.

The effect of consecutive planes at given distances has been partially attacked, and shows that with two planes separated by a distance equal to their breadth, the resistance is hardly 10 per cent. more than for one such plane, and even at seven times that distance the resistance is not doubled.

F. J.

The Auer (Welsbach) Incandescent Gas-Light.

By Professor RENK.

(Journal für Gasbeleuchtung, 1893, p. 321.)

Professor Renk, reporting upon a series of tests made, states that the advantages claimed by the makers of these burners have been fully confirmed. They consume 50 per cent. less gas than argand or slit burners, produce less carbonic acid and heat, do not smoke, and give a light agreeable to the eyes. An Auer burner, lighted almost uninterruptedly for five hundred and twenty-seven hours, consumed 2,430 cubic feet of gas, or an average of 4·6 cubic feet per hour, the consumption varying between 3·8 cubic feet and 5 cubic feet, according to the pressure.

For lighting the lecture-room of the Health Institute, eight Auer burners were tried, and these, with $\frac{30}{10}$ inch pressure, consumed 42 cubic feet per hour, or 5·25 cubic feet per burner per hour. Two slit and five argand burners were also tested, their average consumption was 10·1 cubic feet per burner per hour, which gives a saving of 48 per cent. in favour of the incandescent burners. The contamination of the atmosphere in a gas-lighted room being chiefly caused by carbonic acid and temperature, it may be assumed that the incandescent burners, consuming 50 per cent. less gas, would reduce these objections. An argand burner was lighted in the laboratory; after the amount of carbonic acid in the air had been determined in two positions and the temperature in five positions, and after four hours' burning, the carbonic acid and temperature were again estimated; and the experiment was repeated, under similar conditions, with an incandescent burner. With the argand burner, the carbonic acid was increased from 0·922 to 4·386 per thousand, and the temperature from 37° Fahrenheit to 46·4° Fahrenheit; while with the incandescent burner the carbonic acid rose from 0·946 per thousand to 2·273 per thousand, and the temperature from 34·7° Fahrenheit to 38·6° Fahrenheit. The advantages of the incandescent burners are not limited to these points only, as, in consequence of the complete combustion of the gas, the imperfect products of combustion produced with other burners are almost entirely avoided. With the incandescent burners in use for five hundred and twenty-seven hours in an unventilated room no inconvenience was experienced, while in a better ventilated room with an argand burner lighted during the night a decided inconvenience was experienced. The incandescent light has the further advantage that it does not smoke, however great the gas-pressure may be, the mixture of gas and air being always sufficient for perfect combustion.

Photometrical tests were made to determine the optical value of the incandescent light. With the before-mentioned rates of consumption, slit burners gave an average illuminating power of 14·27 candles, argand burners 29·61 candles, and incandescent burners 55·93 candles. So that, with 50 per cent. less gas, the incandescent burner gives nearly four times more light than the slit burner, and nearly twice as much as the argand burner.

From the hygienic point of view, the brightness, as well as the amount of light, must be considered, that is to say, the amount of light upon a given unit of surface. The brightness of two flames may differ considerably, even when both radiate the same amount of light, in the inverse proportion to the sizes of the flames. As the incandescent burners radiate twice the light of argand burners, and the illuminating surface of the former is about one-half the latter, incandescent burners may be assumed to have four times the brilliancy of argand burners. This is, in one sense, a disadvantage, because it is more necessary to protect the eyes from a brilliant light. This may, however, be easily effected by the use of shades. For this purpose ground glass is preferable to opal, as

the latter absorbs more light. Opal shades give more light immediately under the burner, but not so much as ground glass in the surrounding parts. A steady light is also desirable, and the incandescent light entirely fulfills this condition. It is also eminently suitable for microscopical purposes.

For more than a year the lecture-room of the Health Institute was lighted with four Wenham regenerative burners, the light being thrown upon opaque glass reflectors fixed beneath the burners, being thus partly diffused; but the greater part was reflected from the white ceiling and walls. Two incandescent burners were substituted for each of the regenerative burners, which effected a considerable improvement in the lighting, the increase of light being equal to 121 per cent., while the gas consumed was reduced by 28 per cent.; no smoke was given off, and the light was much steadier.

C. G.

An Inquiry into the Methods of Instruction in Engineering.

By V. A. E. Dwelshauvers-Dery and Julien Weiler.[1]

This volume is the outcome of an agitation, which has been carried on in Belgium by Professor Dwelshauvers-Dery for the past twenty years, for a change in the system of education of young engineers in vogue there, and for the establishment of engineering laboratories in all technical colleges.

The Authors point out in their preface that the purely abstract character of the teaching is not a fault confined to engineering education alone, but that it more or less pervades all professional training.

In order to test the opinion of leading men, both in the engineering and other professions on this subject, a small pamphlet, written by Mr. Julien Weiler, entitled "What the Young Engineer Needs," and a circular, on which replies could be written, were sent out by the Authors. And also a statement which explained their reasons, and stated that the answers would be collected and published in book-form, with an introductory article.

The results of this comprehensive inquiry are given in the actual words of those who replied to the circular, and the volume thus contains a concise summary of the views of all the leading Belgian, and many foreign engineers on the important question of the education of young engineers.

The introduction, by Professor Dwelshauvers-Dery, is largely a reprint from the pages of "Le Génie Civil," of an article on Engineering Laboratories and Technical Colleges, written by the Professor in 1891.

The Author, after pointing out that the need of a laboratory to

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[1] The original is in the Library of the Inst C.E.

aid the instruction of the class-room is just as vital in the teaching of engineering as in the case of physics or chemistry, ascribes their origin to the foundation of the Conservatoire des Arts et des Métiers in Paris, at the close of the last century by the French Republic. The admirable work done by this institution is too well known to all engineers to need recapitulation.

The Author then points out how unanimous the Engineering Congress at Paris, in 1889, was in insisting on the absolute need of properly-equipped laboratories in every technical college and in every institution for the education of engineers. He also quotes the opinions of many distinguished engineers, expressed in letters he had received from them; amongst others—Zeuner, Schroeter, Peabody, Willans, Donkin, Longridge, &c., all of whom heartily supported his views on this subject.

He then gives a brief and concise history of the technical schools, and the system of education for engineers, in the chief European States and America, in particular pointing out how the strict government control on the Continent contrasts with the unrestricted system in England and the States. Attention was drawn to this by the Institution in 1868–70, and in the circular issued by Professor Kennedy prior to the establishment of the Engineering Laboratory at University College, London, in 1875.

A historical sketch of existing engineering laboratories, and of their equipment, is then given; it is shown that the varied equipment for testing materials used by engineers, and for testing the prime movers and other machinery now to be found in all such laboratories, only dates back to 1870. America, in particular, has made wonderful strides during this period, and her crowded technical colleges, all equipped in the most complete manner and on a scale almost unknown in Europe, testify to the want they have supplied, and to the value the practical people of that country attach to a thorough and systematic education of engineers, and the part they consider the laboratory and workshop play in that education.

The Author points out that in the continental laboratories the work carried out has mostly been extensive research by the teaching staff, with demonstrations only to students; while in England and America, on the contrary, individual work for each student has mainly been arrived at, so that the student himself might be trained by actual working at the various machines in habits of experiment.

Many of those who supported the Authors' views, point out the twofold character of such laboratories, namely, their need for the student and their value as places in which original research can be carried out, and the many problems so often confronting engineers be solved. To the student the laboratory makes that which was abstract concrete; it fixes in his mind the dry facts of the class-room, it trains him to habits of exactness and accuracy, and fits him for positions of trust and responsibility. To the professor it gives the opportunity of dealing with problems of surpassing interest,

which, from the pressing demands of business, can of necessity be undertaken by few engineers engaged in active professional work.

The laboratory, in short, enables Schroeter's maxim to be carried out—"very much less lecture work and very many more practical exercises, experiments in the laboratory and individual study of the results of those experiments."

After the introduction comes the letter of Mr. Weiler on the system of engineering education in Belgium; he declares that while the students are well equipped with a mass of theoretical knowledge they have not the least practical acquaintance even with the form of the machines and structures they have been studying, or with the actual practical working of the various motors whose cycles they have so diligently theorised. He deplores this state of things, and strongly advocates a complete change and the adoption of laboratories and workshops in all their technical institutions.

A copy of the circular which was sent out is then given, and a list of those engineers who supported it.

The rest of the volume, some three hundred pages, is devoted to the replies received to this circular, and to extracts from the press articles in which it was noticed.

The answers are divided into groups. The first group contains replies from engineers, chiefly, of course, Belgian, but also many well-known French, German, and English engineers, some only expressing in a few words their complete approval of the ideas of the Authors, but others writing very fully their views on this important subject. These replies will therefore be found of great interest by any one who will carefully read them; there are no less than three hundred and nine given.

The second group gives in a similar way the opinion of those who, though not themselves engineers, are, from their position and training, fitted to express opinions of value on such a subject. Fifty-six such replies are printed, most of them from eminent Belgians in every walk of life.

The third group summarizes articles which appeared in twenty-nine Belgian technical and daily newspapers; most of them, as is the custom with the continental press are signed articles, and are complete criticisms of the pamphlets and circulars, and are, therefore, of great value as giving the drift of general public opinion on the education necessary for engineers.

The last thirty pages are devoted to a bibliography of the subject of engineering laboratories, which is very complete and of considerable use to those interested in this question.

T. H. B.

Regulations governing the Issue of Survey Licences in the State of Perak.

(Perak Government Gazette, July 21, 1893.)

The following rules for the licensing of surveyors, drawn up by the chief surveyor, have received the approval of Government and are published by authority, and will, from January 1894, supersede the existing regulations.

1. Examinations for the issue of licences to practise as land surveyors will be held at the Trigonometrical Survey Office, Taiping, twice a year, during the months of April and September, of which not less than one month's notice will be given in the *Government Gazette.*

Each candidate desirous of being examined must give written notice to the chief surveyor, stating which class of certificate he wishes to compete for, and deposit the examination fee at the Survey Office, Taiping. Such notice must be received by the chief surveyor not less than seven clear days previous to the date fixed for the examination to commence, and must be accompanied by documentary evidence, (*a*) that he is over twenty-one years of age, (*b*) that he is a person of good character.

2. Two licences will be issued—namely, a first- and second-class licence. The fee for the former will be $15, and for the latter $5. The holder of a first-class licence will be authorised to undertake any survey work in the State of Perak. The holder of a second-class certificate will not be allowed to survey on his own responsibility, but may act as an assistant to the holder of a first-class licence.

3. (*a*) For a first-class licence, the candidate, before entering upon his examination, must produce documentary evidence of having been employed for not less than two years in the field, and one year in an office with a qualified surveyor or surveyors. If he has been employed by or with a qualified surveyor in the Straits Settlements, he must produce a certificate of competency to undertake surveys on his own responsibility. Each candidate is required to produce one of his own original field-books and plans plotted therefrom, of which plans one at least must represent an area of not less than 40 acres, with a river or road frontage, showing as much topographical and detail information as possible.

(*b*) For a second-class licence the candidate must have been actually employed at least one year in the field, on *bonâ fide* land surveys, in the Straits Settlements, and have been at least six months in a surveyor's office, and he must produce an original field-book and work plotted therefrom, together with a certificate from a qualified surveyor in the Straits Settlements certifying to his competency.

4. *Syllabus for First-Class Certificate.*—Candidates will be examined in the elements of plane geometry, trigonometry, logarithmic computation, calculation of areas and quantities, and will

be required to work out a traverse by Gale's method; elements of astronomy as applied to the methods of determining the latitude, time, and true meridian; the uses and adjustments of the prismatic compass, plane table, box sextant, theodolite and dumpy level, and the principles of their construction. An instrument out of adjustment will be given to each candidate to be adjusted.

The candidate will be further examined in the general principles and practice of surveying applicable to the Straits Settlements, and the methods employed in exact lineal measurement, having regard to variations of temperature and errors of calibration. Also the various methods of determining heights, viz., by zenith distances, barometer and boiling-point thermometer. The text-books recommended are "The Indian Manual of Surveying," "Raper's Astronomy," "Todhunter's Euclid, Mensuration and Trigonometry for Beginners," and Thomason College Text-Books on Surveying.

Second-Class Certificate.—For a second-class certificate the candidate will be examined in plane trigonometry, logarithmic computation, calculation of areas and quantities, Gale's traverse system, the uses and adjustments of the prismatic compass, plane table and theodolite, and the principles of their construction. And the candidate will be further examined in the general principles of surveying as applied to revenue surveys in the Straits Settlements. The text-books recommended are the Thomason Text-Books.

All candidates will be required to make and plot a survey which will be pointed out to them when they come up for examination. They will be required to supply their own instruments.

5. The examination will be three days for field work and plotting, and two days for written work and adjustments.

For the purpose of marks, the examination for a first-class certificate will be divided into three sections:—

(i.) Field-work and theory and adjustments, 100 marks.

(ii.) Mathematics, 100 marks.

(iii.) General principles of surveying, and astronomy as applied thereto, 100 marks. Candidates must score at least two-thirds of the maximum number of marks in each subject to pass.

For second-class certificates the marks will be:—

(i.) Field-work, theory and adjustment of instruments, 100 marks.

(ii.) Mathematics and principles of surveying, 100 marks.

At least two-thirds of the maximum number of marks in each section must be obtained to secure a pass.

6. *General.*—Certificates will be granted by the chief surveyor to all candidates for the class they pass, but this will not entitle them to practise as land surveyors until their certificates have been registered in the office of the Secretary to Government and published in the *Government Gazette*. Such certificates to be held during good behaviour only, and may be cancelled by notification in the *Government Gazette* at any time, should the Government consider it expedient in the public interest so to do, and after the

licensee has been called upon to show cause why his licence should not be cancelled.

The chief surveyor may grant a certificate without examination to any applicant who has passed a recognized examination as a land surveyor of an equal standard to that laid down for Perak in any part of the British Empire, or who has been employed as a first-class assistant surveyor in the service of the native States or the colony of the Straits Settlements, but he may at all times require a candidate to undergo examination if he thinks it desirable to do so.

Applicants who have been duly trained as Civil Engineers and have been employed as field assistants for a period of at least one year may be considered, at the discretion of the chief surveyor, to fulfil the conditions as to field experience laid down in paragraph 3.

INDEX

TO THE

MINUTES OF PROCEEDINGS,

1893-94.—Part I.

N.B.—Titles in italics refer to Original Papers, and those selected for printing only are further distinguished by the suffix "(S.)." Abstracted Papers are not so indicated.

LONDON: PRINTED BY WILLIAM CLOWES AND SONS, LIMITED, STAMFORD STREET AND CHARING CROSS.

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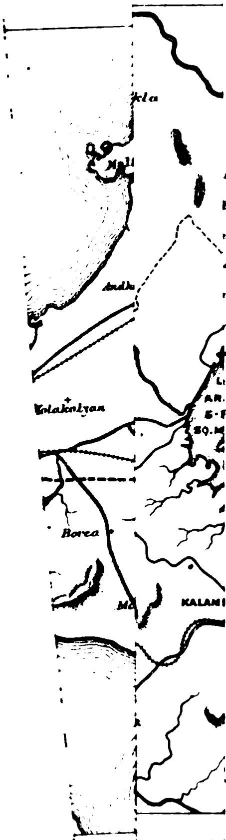
BAY

PLATE I.



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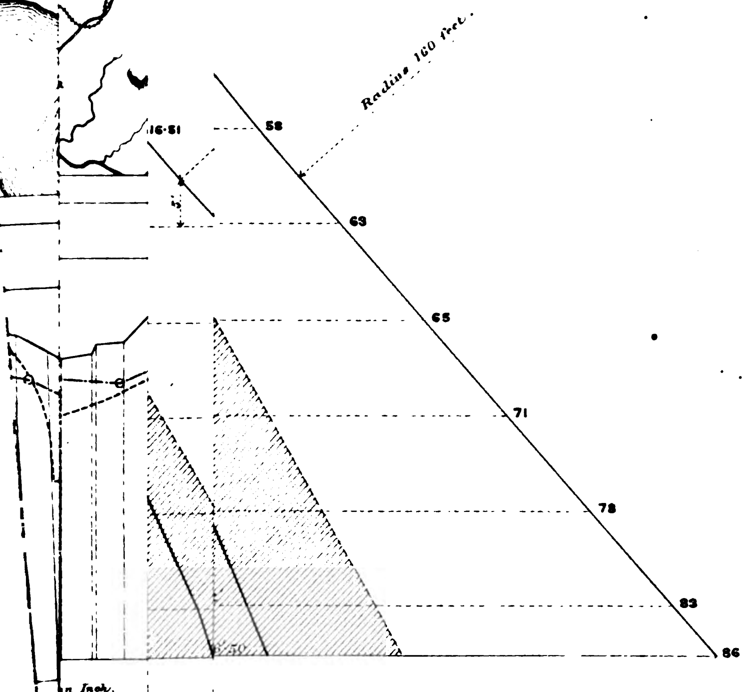

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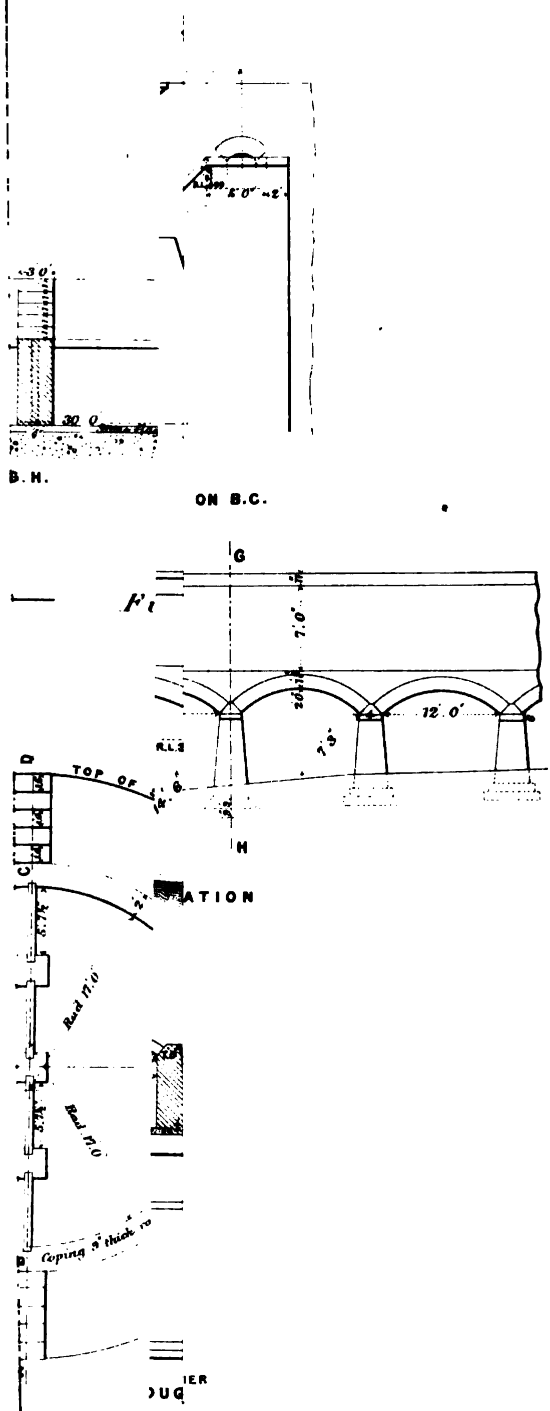
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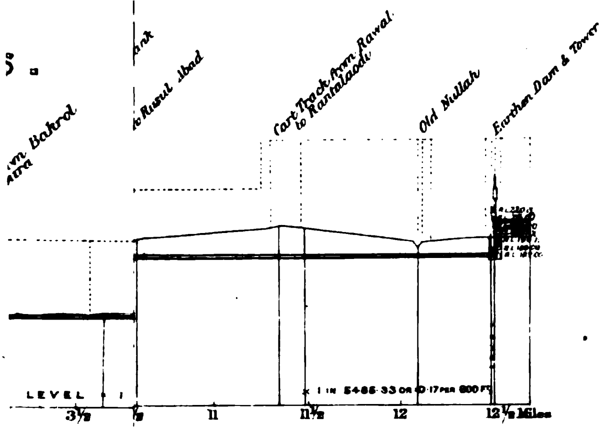

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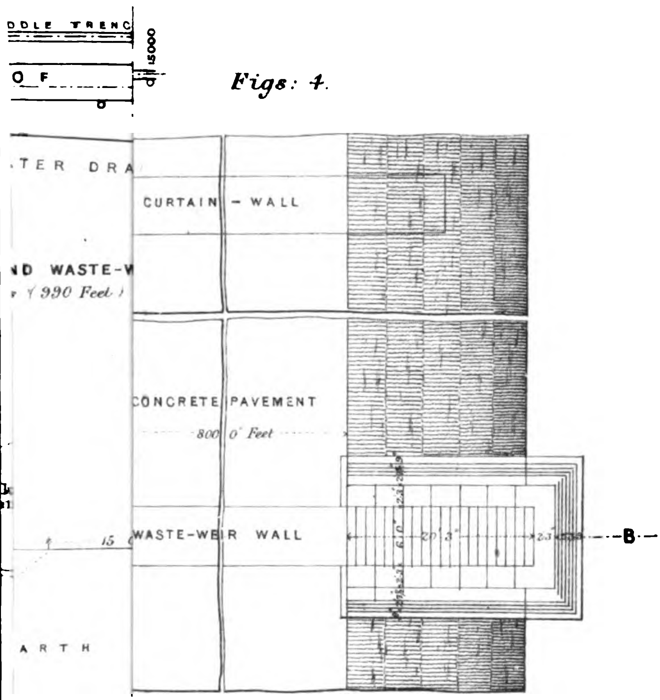

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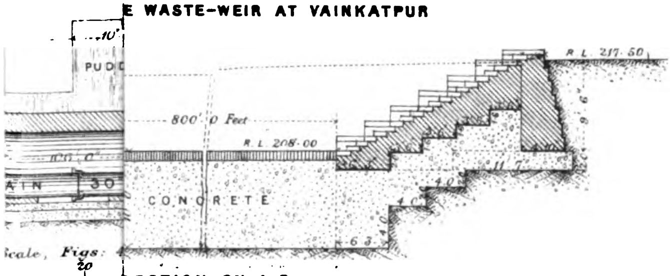
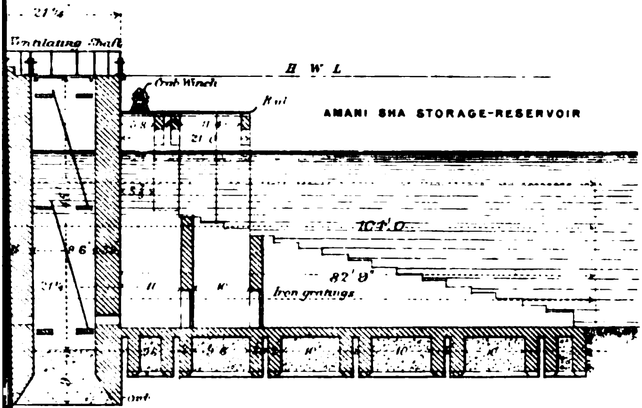


SECTION ON A.B.

THOS KELL & SON, LITH. 40, KING ST COVENT GARDEN

PLATE. 4.



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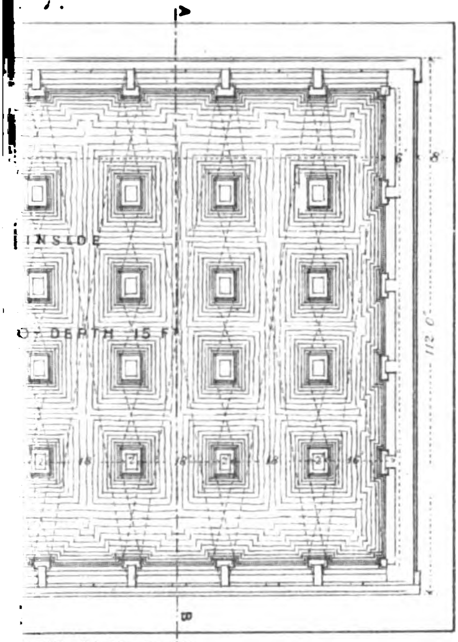

VOIR, PART PLAN.

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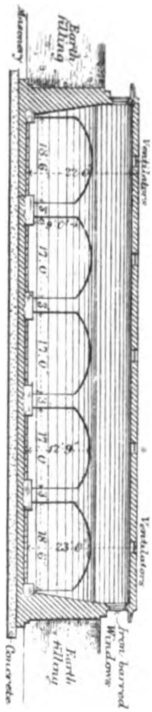

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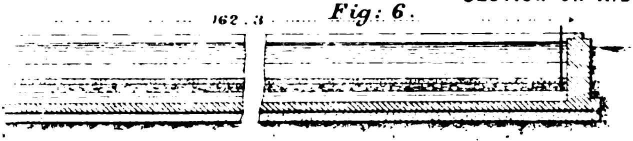

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10 · 20 · 30 Feet

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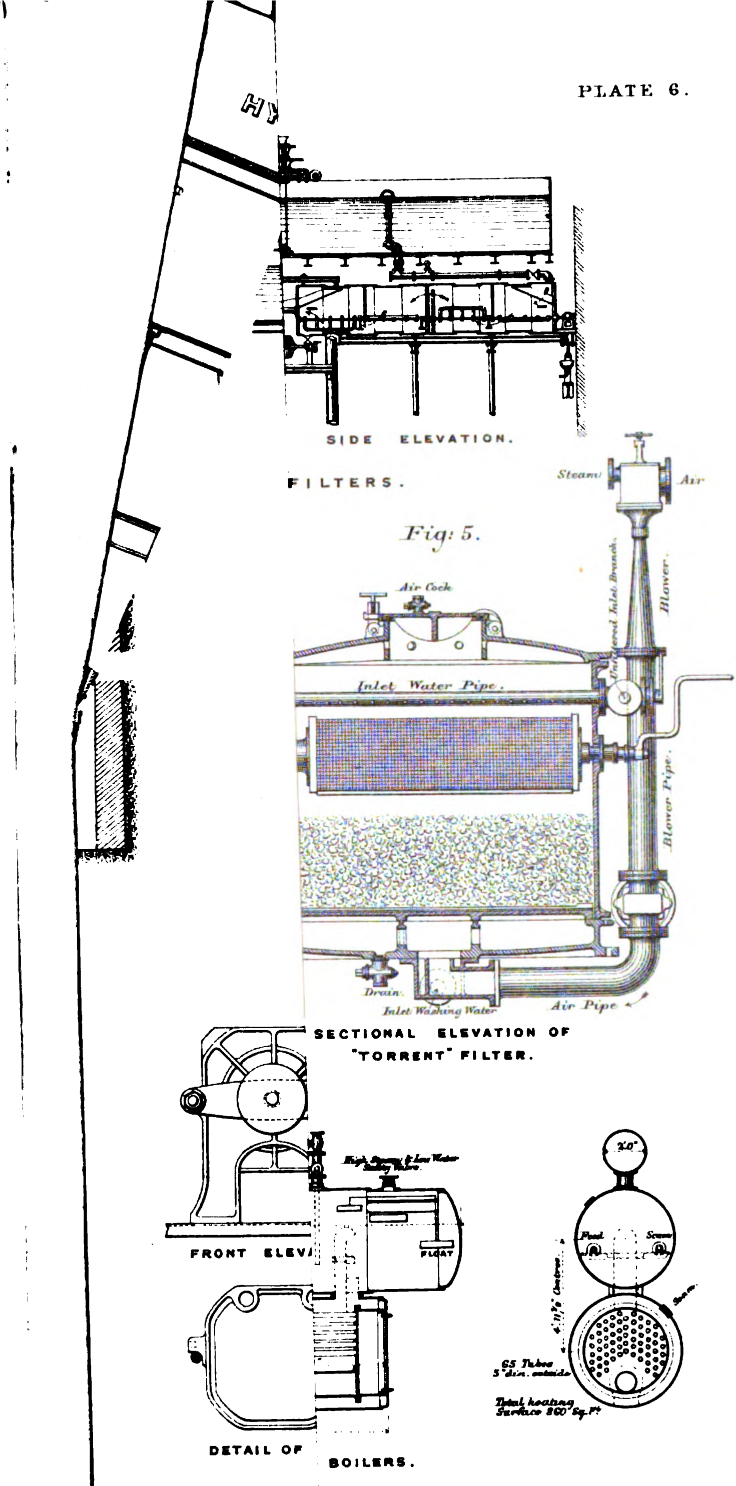

SIDE ELEVATION.

FILTERS.

Fig: 5.

SECTIONAL ELEVATION OF "TORRENT" FILTER.

FRONT ELEV

DETAIL OF BOILERS.

THOS KELL & SON, LITH. LONDON

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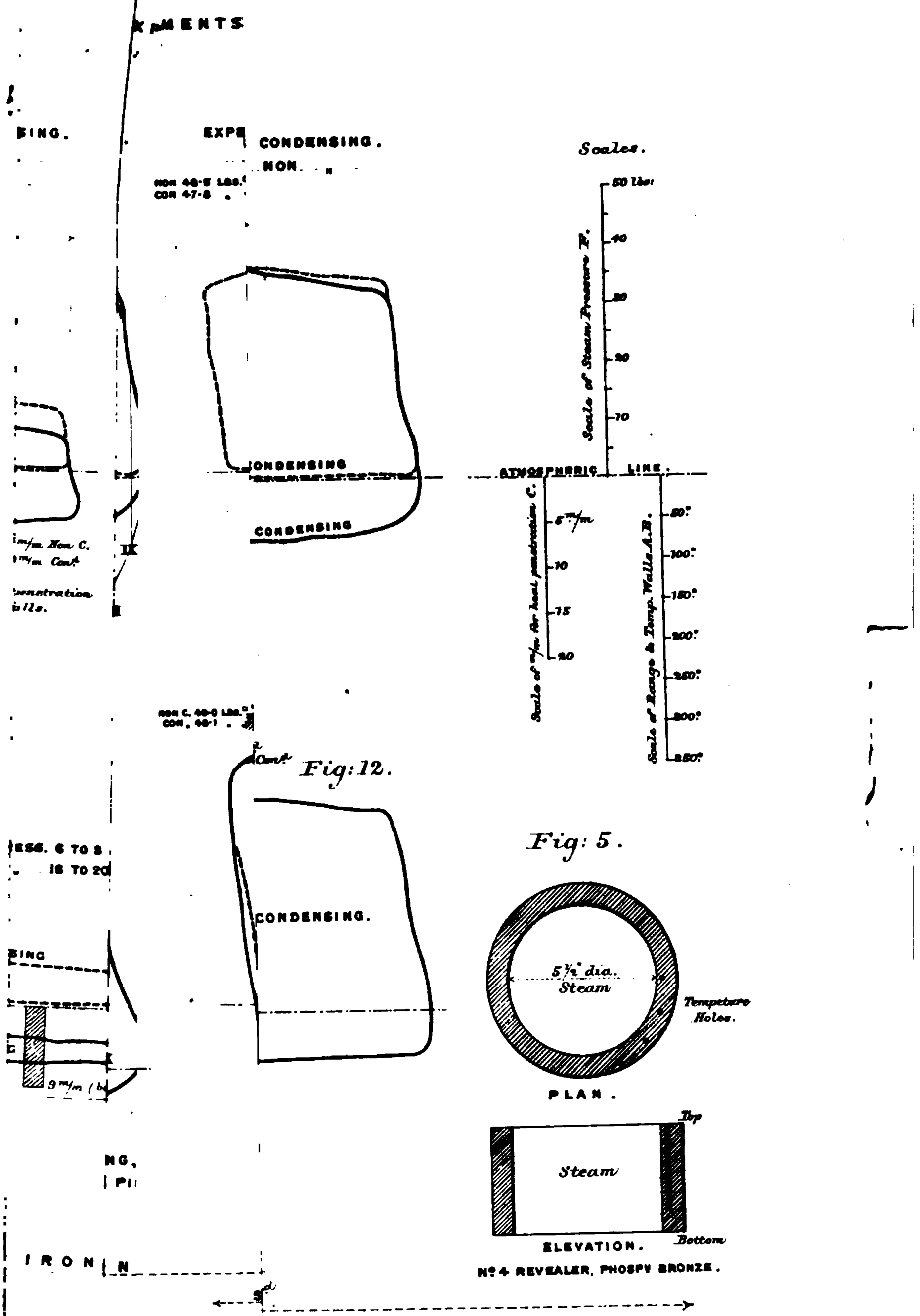
PLATE 7.



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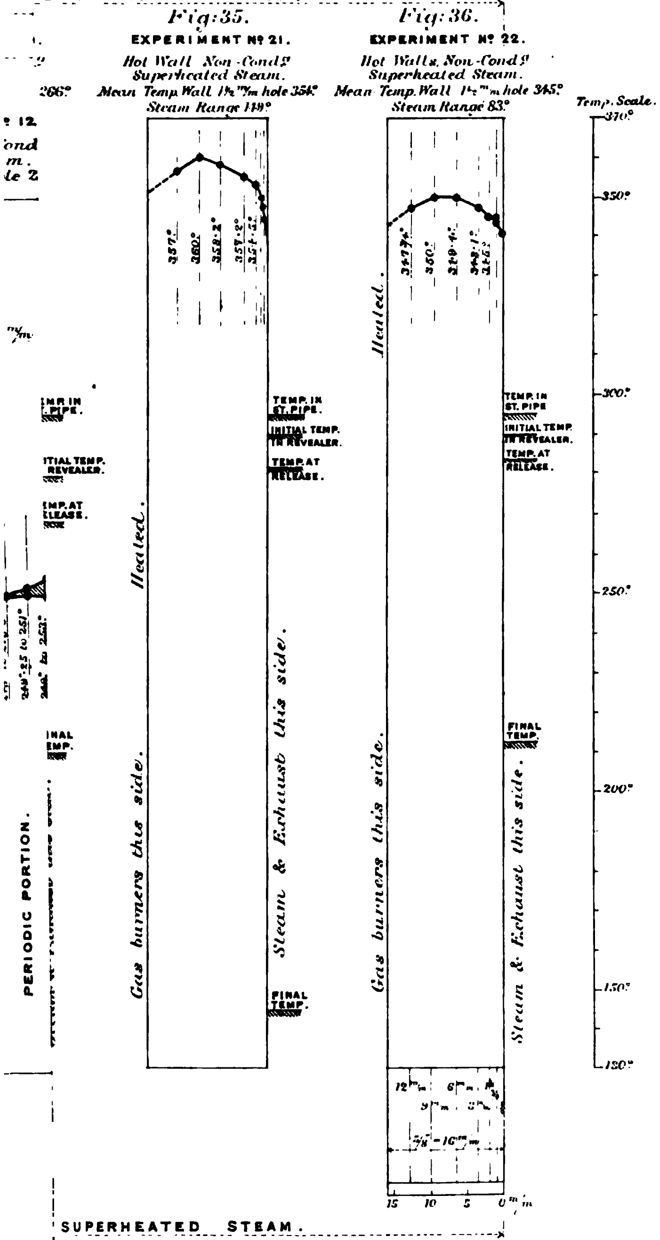

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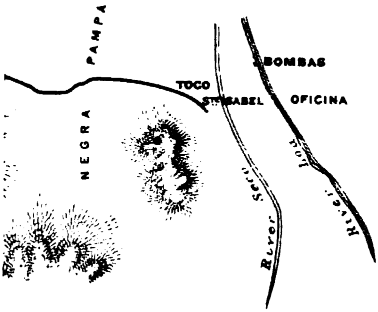

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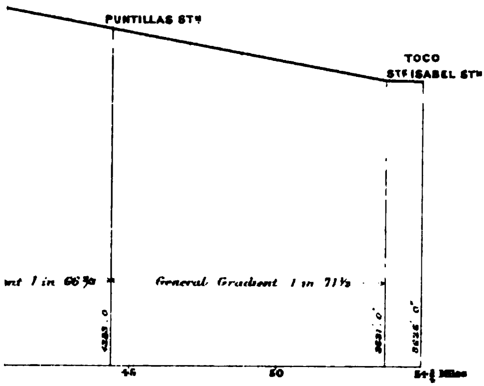

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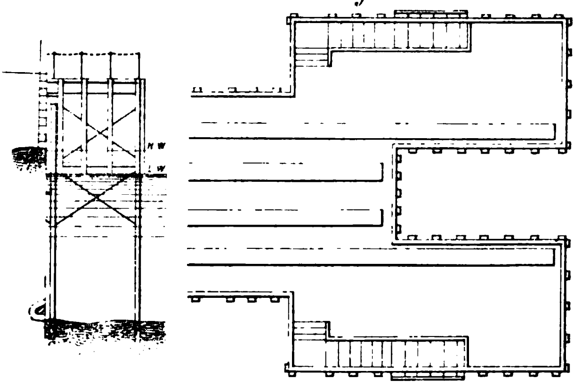

PART PLAN.

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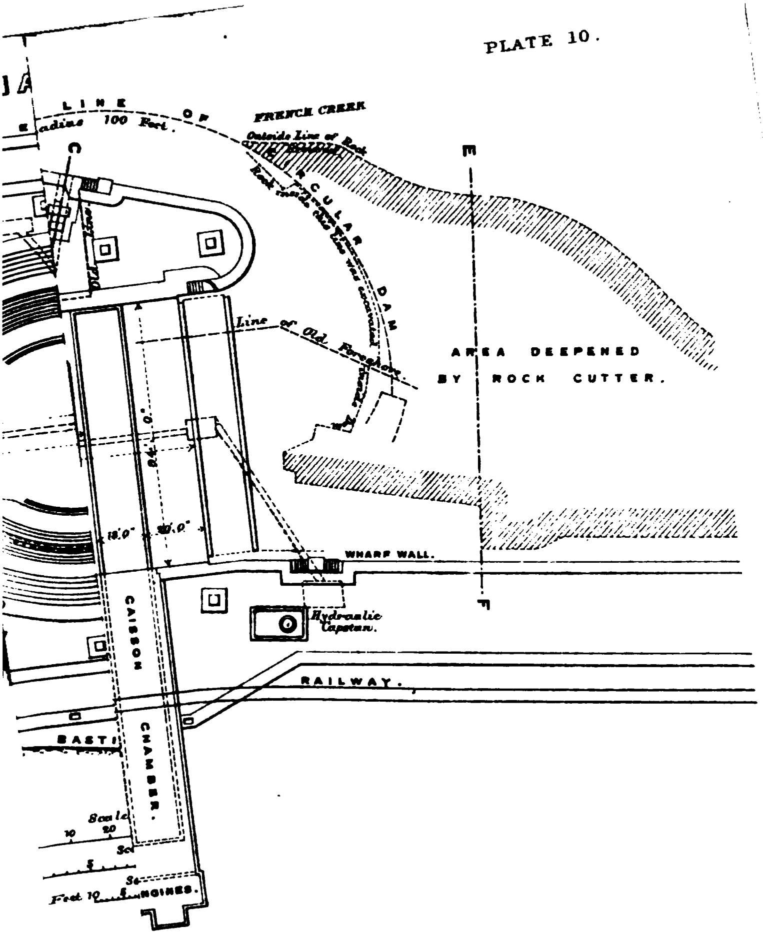
PLATE 10.



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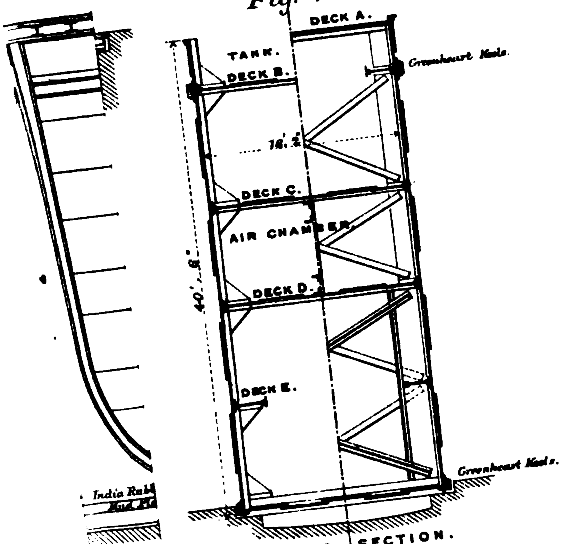

CROSS SECTION.

THOS KELL & SON LITH LONDON.

Minutes